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CIVIL ENGINEERING

SEP 3 1931

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American Society of Civil Engineers*

Albany Falls, Clark Fork River
Site of Diversion Dam for Columbia Basin Project, Gravity Plan

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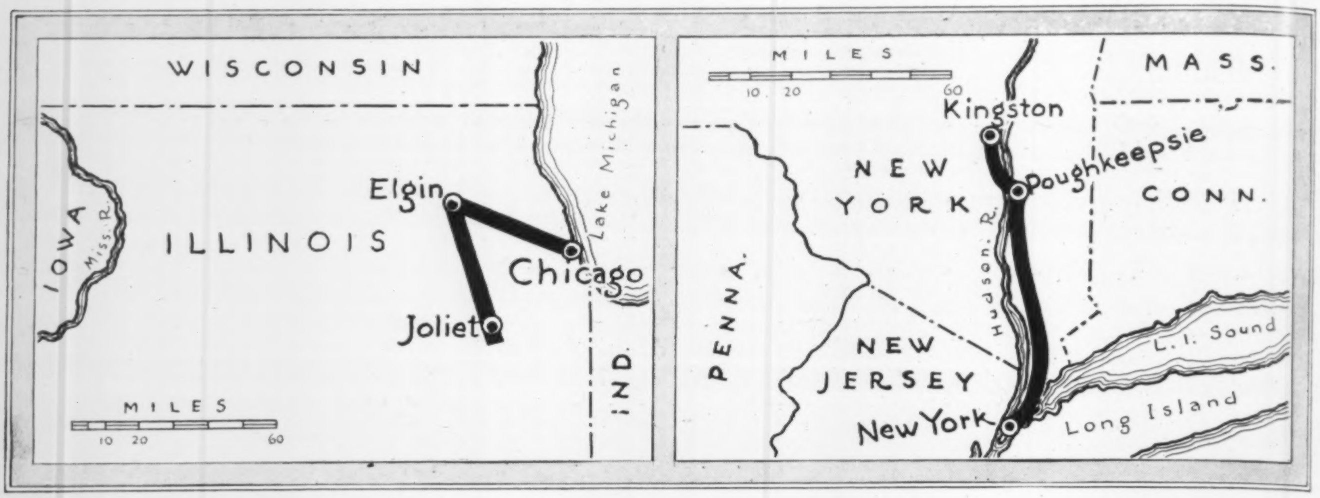
SEPTEMBER 1931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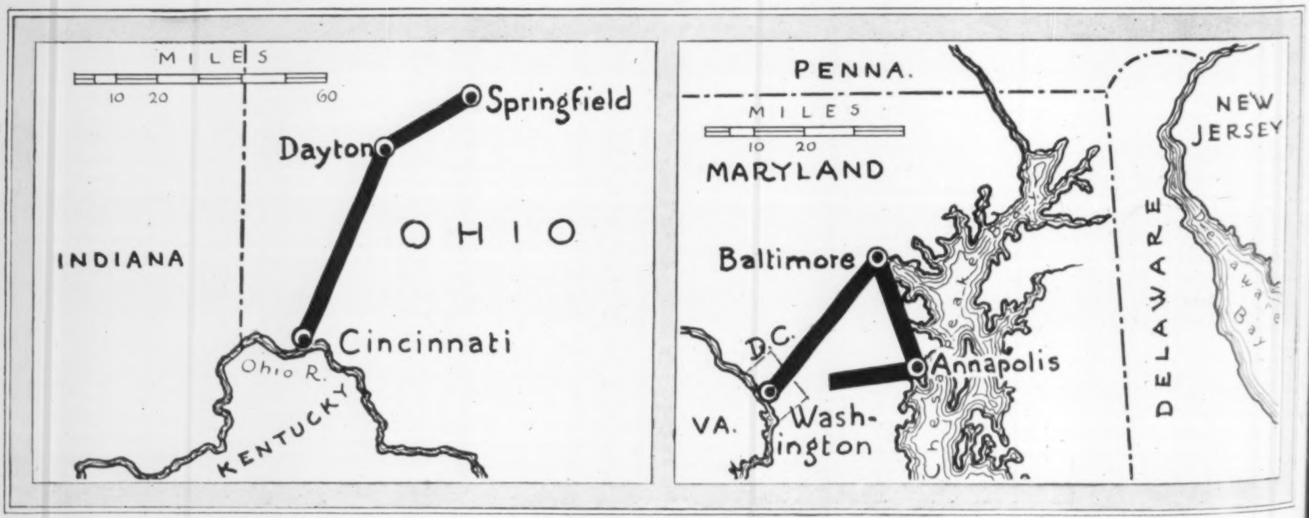
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VA.
Washington
DELAWARE
Delaware Bay

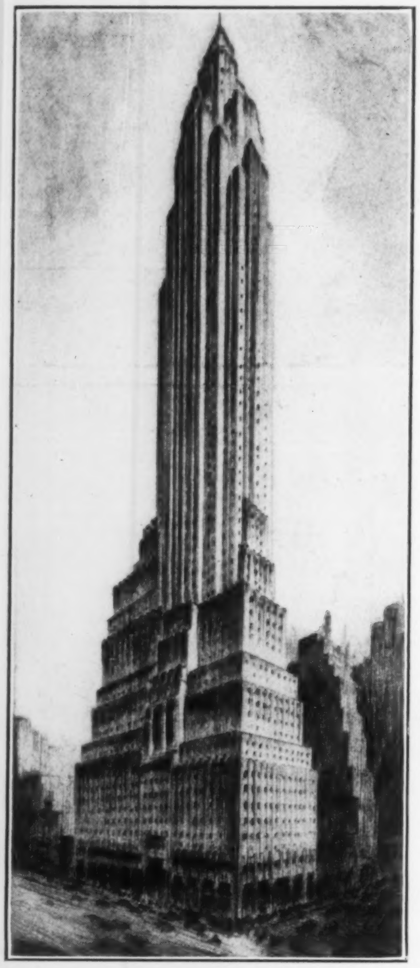
St. Paul Sky Line

Great Northern Railroad Bridge

St. Paul Fall Meeting

AMONG the cities which have developed from early Indian camping grounds is St. Paul, which was first visited by that courageous Jesuit missionary, Father Hennepin, in 1680. Incorporated as a city in 1854, this seat of government of the great state of Minnesota, together with its sister city of Minneapolis, has a population of three quarters of a million people.

ST. PAUL MEETING
OCT. 7-10, 1931



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Industrial Plants - Warehouses - Railroads and Terminals - Foundations - Underpinning
Filtration and Sewage Plants - Hydro-Electric Developments - Powerhouses - Highways
River and Harbor Developments - Bridges and Bridge Piers - Mine Shafts and Tunnels

Among Our Writers

Ralph Budd, who is well known for his work in directing numerous great railroad enterprises, recently headed a commission of experts to advise the Soviet Government on the reconstruction and expansion of the railroad systems of Russia.

John S. Butler entered the Corps of Engineers at the outbreak of the War after a long experience in river improvement and lock construction.

Ross K. Tiffany, an authority on water works engineering, was connected with the development of the Yakima Irrigation Project for 20 years.

D. B. Steinman specializes in building long-span bridges. He has been identified with the design and construction of such notable structures as the Florianopolis, Carquinez, Mount Hope, Hell Gate, and Triborough bridges.

L. Murray Grant was building pipe for irrigation projects as early as 1909. He was chairman of the Board of Public Works and Superintendent of the Water Department of Seattle from 1926 to 1928.

T. G. McCrory, who is considered an authority on the construction and maintenance of state roads, has occupied various important positions in the Washington State Highway Department.

J. C. Stevens, inventor of the Stevens water level recorders and indicators, served for 8 years in the U.S. Bureau of Reclamation and the U.S. Geological Survey.

L. Standish Hall has been in charge of groundwater investigations on the proposed Mokelumne water supply project since 1924.

Fred C. Scobey, for nearly 20 years a specialist in experimentation and research on the carrying capacity of water conduits and structures, is the author of numerous works on this phase of hydraulics.

C. E. Putnam, a Minnesotan by birth, went to the Pacific Coast in 1900. There he has had charge of the construction of many railroads, irrigation projects, industrial plants, and port facilities.

Verne Gongwer has been employed by the City of Tacoma since 1923. He designed and constructed the record 6,241-ft. span transmission line which crosses the Narrows.

T. H. Carver has served continuously in the City Engineering Department of Seattle for the past 26 years. The construction of all important water supply and distribution systems during this period has been under his supervision.

William D. Shannon has been engaged on such important hydro-electric projects as the Big Creek and Caribou developments in California and the White River, the Baker River, and the Rock Island developments in Washington.

Lyman Griswold began his engineering career in 1898 and for the past 10 years has concentrated his efforts on hydraulic investigations.

William J. Fox's engineering experience includes highway construction in Hawaii and work as field engineer for commercial organizations in the Philippines and Peru. Since 1922 he has been identified with California projects.

E. E. East, has headed two recent expeditions to explore and map the route of the International Pacific Highway through Mexico to San Salvador, Central America.

L. I. Hewes has combined a career of teaching at Rhode Island State College and Yale University with work for the U.S. Bureau of Public Roads, with which he became associated in 1911.

W. W. Crosby, for 8 years Chairman of the Society's Special Committee on Road Materials, commanded the 104th Engineers in France.

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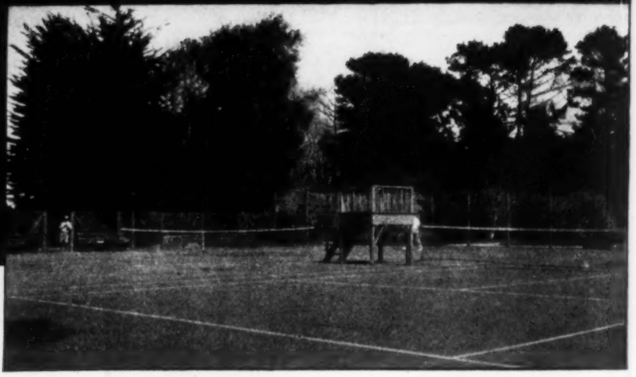

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September 1931

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Developing the Pacific Northwest

Engineers Build a Modern Civilization in a Virgin Country

By Ralph Budd
Member American Society of Civil Engineers
President, Great Northern Railway, St. Paul, Minn.

THAT part of the continent which lies beyond the Rocky Mountains, and north of California, Nevada, and Utah, is known as the Pacific Northwest in order to distinguish it from the more loosely defined region west of the Mississippi, or from the Northwest Territory. No part of the United States has a more romantic or piquant historical background than Old Oregon, which includes Oregon, Washington, Idaho, and a part of Montana and Wyoming. From the very beginning of the period when definite facts took the place of imagination concerning this region, the engineer, using the term in the broadest sense, has had a leading part in its development. Speaking before the Sixty-first Annual Convention of the Society at Tacoma, on July 8, Mr. Budd presented a dramatic picture of the rapid progress that has been made in the Pacific Northwest. His address, here abstracted, is a fitting foreword to this issue, which is devoted exclusively to the papers presented at the Convention.

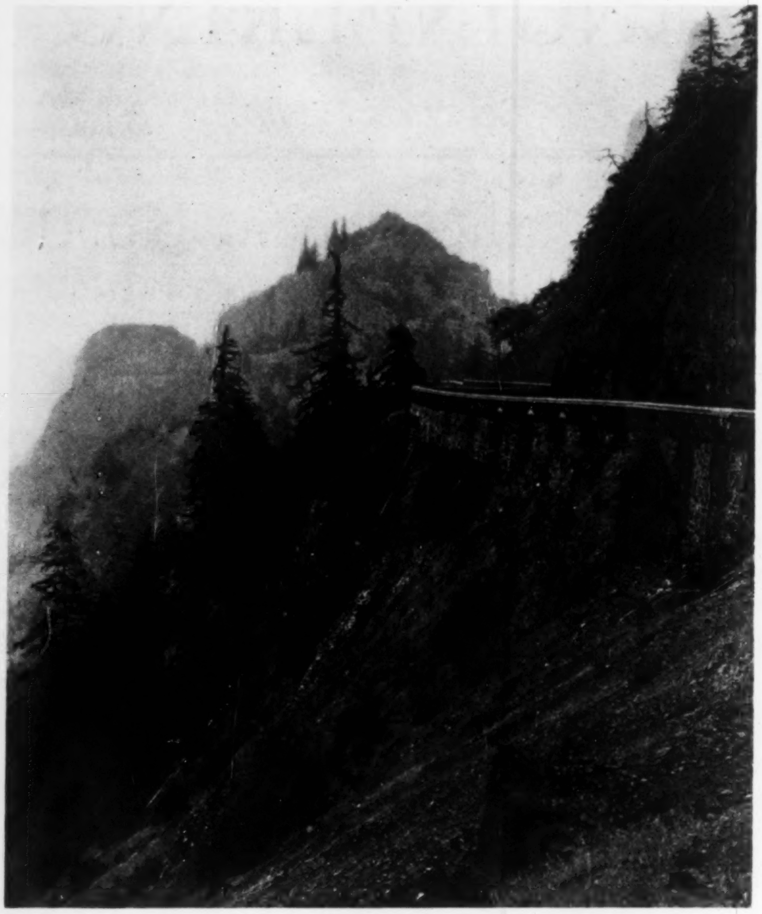
IT WAS not until after the British Captain Vancouver, in the sloop *Discovery*, and Broughton, commanding the armed tender *Chatham*, surveyed and mapped the shores of Puget Sound, that the idea of a water route through or around the American continent somewhere between the Isthmus of Panama and the Arctic Ocean was dispelled and dreams of the Northwest Passage faded. Nearly all of the early maps suggested the possibility of such a route, and the lack of accurate information about the entire country west of Hudson Bay and the Great Lakes provided an excuse for filling the vacant spaces with channels, seas, and lakes to fit the fanciful tales which had persisted from the time when the idea of finding a westward route to India prompted Columbus to set out on his epochal voyage of discovery.

As the shores of the continent took definite shape on successive maps, due to the knowledge obtained by such explorers as the Cabots, Cartier, Cabrillo, Drake, Hudson, Bering, Cook, Quadra, and many others, little remained to the imagination except the regions north of California and south of Alaska. Considering the early date at which the western shore of California was mapped, the northwest coast of the present United States remained unknown to a very late date. As late as the year 1797, before the surveys of Vancouver became available through the publication of his data, the best maps were very indefinite as to the coast line of Oregon and Washington, while some confidently indicated a huge sea or bay as large as the State of Washington opposite the Strait of Juan de Fuca. The mythical Straits of Anian led indefinitely northeastward to Hudson Bay or the Arctic Ocean.

At the same time that Vancouver was beginning his monumental survey of Puget Sound, Capt. Robert Gray of Boston discovered and partly explored the great river which he named for his ship, *Columbia*. That was in May 1792. The work of these two men and of Broughton, who, under directions from Vancouver, extended Captain Gray's explorations up the Columbia, constituted the earliest contribution of the engineer to the Pacific Northwest. The English names which predominate in this

Oregon City Woolen Mills

region, such as Puget Sound, Vancouver Island, Hood Canal, Whidby Island, Mount Hood, Mount Baker, and many others—among them that of Rainier, which never successfully displaced the euphonious Indian name "Tacoma" for the mountain which dominates the Puget Sound region—were given by Vancouver. These names remain to commemorate the great initial surveying venture into this country, which was by sea.



Columbia River Highway, Near Portland, Ore.

NORTHWEST OBTAINED BY DISCOVERY

In point of practical results affecting international policies and relations, Gray's discovery of the Columbia River was of even more importance than Vancouver's surveys. Upon that event, supplemented and supported by the overland expedition of Lewis and Clark in 1805, and the commercial occupancy of John Jacob Astor at Astoria in 1811, the United States based her successful claim of sovereignty over most of the Old Oregon Country, which is now called the Pacific Northwest. In this way, as is often pointed out, the United States obtained by virtue of discovery, exploration, and occupancy nearly one-tenth of the present area of the country; and that one-tenth, moreover, is the only part of the entire nation which was not obtained by conquest of arms or by purchase.

Not until 1846 was it finally decided that the forty-ninth parallel of latitude should form the boundary between the United States and the British dominions west of the Rocky Mountains. So it happened that only 85 years ago all of this vast country was a virgin land, practically unoccupied except by Indians. Consequently its whole development has taken place during a period characterized as the "Machine Age" because of the rapid advancement and universal application of scientific knowledge and invention.

The very rugged topographical features of the Pacific Northwest presented at once unprecedented obstacles and unequalled opportunities for the engineer. In fact, the success of the railway as a transportation agency and the feasibility of operating railways on fairly steep grades through mountainous country were really the determining factors in making the Pacific Northwest an integral and homogeneous part of the Union. This factor also made possible its phenomenal growth in population, wealth, and industry.

After the Lewis and Clark expedition, the most important early engineering undertaking was the exploration known as the Pacific Railway Surveys, carried out by order of Congress in 1853 and 1854. Isaac I. Stevens, later the Governor of Washington Territory, had charge of those surveys in the Pacific Northwest. Simultaneously with his activities in the north, three other routes were surveyed farther south. The result of these surveys, published in 13 volumes in 1855 as Senate and House documents, contains, in my opinion, more important source material about the West than is found in any other public or private report.

The Northern Pacific reached Portland and Tacoma in 1883, and in 1884 the Oregon Short Line was connected up. In 1887 the Northern Pacific crossed the Cascades from the confluence of the Columbia and Snake rivers at Pasco, and in the same year the Oregon and California Railway was built over the Siskiyous to join the Southern Pacific, thus making a continuous line from Portland to San Francisco. In January 1893, the Great Northern reached Puget Sound, and in 1909 the Chicago, Milwaukee, and St. Paul Railway completed its line from the Missouri River at Mobridge to the Pacific.

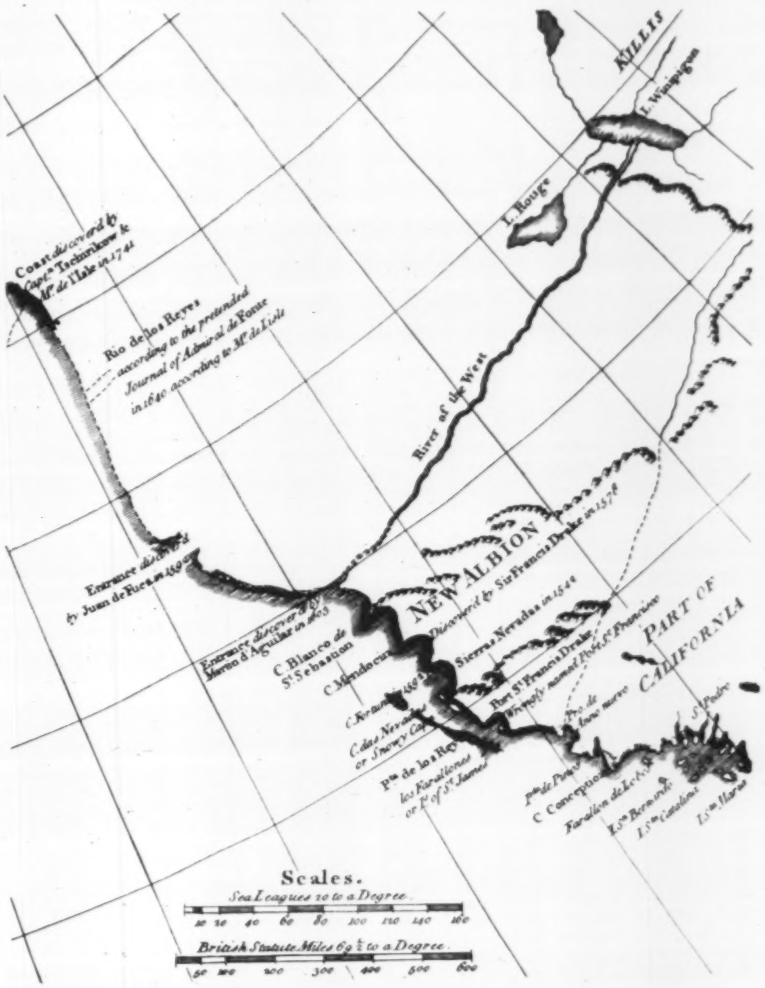
Among the many who had a noteworthy part in building the early railways into the Pacific Northwest are the late Grenville M. Dodge, Hon. M. Am. Soc. C.E., and the late W. Milnor Roberts and Virgil G. Bogue, Members Am. Soc. C.E. All achieved greatness beyond the limits of the Pacific Northwest, but here they were active in their prime—Mr. Dodge in building the Union Pacific, Mr. Roberts and Mr. Bogue in building the Northern Pacific.

THE BUILDING OF CITIES

The rapid influx of population into the Northwest, following railway construction, called for the creation of all of the municipal improvements that are identified with the modern city, and in the case of the seaports, the problem of creating the necessary harbor facilities was also present. Promotion plans were put forward and some visionary projects were advocated, but it is noteworthy that, in the main, future growth was accurately appraised by the early planners. Among those who believed that terminals along Puget Sound would be extensive was Virgil Bogue. All the pioneer engineers seem to have believed that there would be a large city at Tacoma and another at Seattle, and they set about providing for these eventualities.

It is worthy of note also that the harbor developments at the south end of the City of Seattle were carried out according to Virgil Bogue's original conception for improving the mouth of the Duwamish River, and that the late Hiram M. Chittenden, M. Am. Soc. C.E., planned the Lake Union locks as well as the port improvements in the northern part of Seattle.

While no one man to my knowledge can be singled out for special mention in connection with them, modern improvements calling for distinguished engineering services have been carried on in Tacoma Harbor and in the Columbia and Willamette rivers. This work included building the jetties which removed the Columbia bar and made of Portland a safe and accessible seaport. Ports in the Pacific Northwest which have been improved by dredging include Portland, Astoria, Longview, Vancouver, Coos Bay, Umpqua River, Seattle, Tacoma, Olympia, Everett, Anacortes, Bellingham, Port Angeles, Grays Harbor, Willapa Harbor, and others. Much of this work has been done under the direction of the Army Engineers, and the cost, which was borne by the U.S. Government, amounted to $65,000,000. It was more than a coincidence that the need for improved dredging on the Pacific Coast came exactly at the time when modern dredging methods were being evolved. The work here gave impetus to the further perfection of the science of dredging.


KILLIS
L. Winipigon
L. Rouge
Coast discovered by Captn. Tschirikow & Mr. de l'Isle in 1741
Rio de los Reyes according to the pretended Journal of Admiral de Fonte in 1640 according to Mr. de Lisle
River of the West
Entrance discovered by Juan de Fuca in 1592
Entrance discovered by Martin d'Aguilar in 1603
C. Blanco de St. Sebastian
C. Mendocino
NEW ALBION
Discovered by Sir Francis Drake in 1578
Sierras Nevadas in 1544
Port St. Francis Drake
Wrongly named Port St. Francisco
C. Fortunas 1593
C. das Nevadas or Snowy Cape
Pta. de los Reyes
los Farallones or Is. of St. James
PART OF CALIFORNIA
Pro de Ano nuevo
Pta. de Pinos
C. Conception
Farallon de Lobos
Isa. Bernardo
Isa. Catalina
Isa. Marcus
St. Pedro
Scales.
Sea Leagues 20 to a Degree
10 20 40 60 80 100 120 140 160
British Statute Miles 69½ to a Degree
50 100 200 300 400 500 600

From a Map Published by the Imperial Academy of St. Petersburg About 1775

In Striking Contrast to the West Coast of the American Continent, Siberia Had Been Extensively Explored and Mapped at This Time

Sky-Line Logging on the Duckbush River, Wash.

Precipitous Slopes and Enormous Size of Trees Make Ordinary Methods Impossible

Water supply problems for large cities in the Pacific Northwest have been comparatively simple because of their nearness to sources of pure mountain water. The precipitous character of the Cascade Mountains and the solid rock generally encountered have, however, made necessary some expensive works. Probably no cities in the world have been more adequately supplied with pure water than have all these coast cities, as well as Spokane, Yakima, Wenatchee, and others in the interior. A significant and typical feature of the plans of these water supplies is that, although they were made when the cities were in their infancy, they provided for a future growth to several times the present population.

The mountain sources of water for irrigating the valleys of the foothills and the more distant plateaus have been put to beneficial use in the Pacific Northwest by three groups of engineers: first, those representing private capital; then those who represented the states, functioning under the Carey Act of 1894; and finally, the engineers of the Federal Reclamation Bureau, which was organized by congressional enactment of 1902. How early in the history of the Northwest white settlers began to irrigate is shown by the fact that the Mormons built canals in 1849. This is said to be the first large-scale irrigation organized in the United States.

Some of the reclamation which was begun as a private venture was later included in Government projects.

In some instances, reclamation has consisted of drainage instead of irrigation; in others—notably in the Klamath Project in southern Oregon—there is both drainage and irrigation; and in still others—for example at Minidoka, Idaho—hydro-electric power is generated in connection with irrigation. Some of the reclamation in the Pacific Northwest has involved such large engineering works as the Arrowrock Dam near Boise, Idaho. Outstanding among great projects of the future is the irrigation of the Columbia Basin in eastern Washington, where nearly 2,000,000 fertile acres lie waiting the magic touch of water.

WEST PORTAL OF THE EIGHT-MILE CASCADE TUNNEL
Great Northern Railway

HYDRO-ELECTRIC POWER DEVELOPED

The modern hydro-electric power plant is of such recent origin that much of its evolution has taken place in the Pacific Northwest. Here the mountains and streams are so related that dam sites, power sites, and stream-flow conditions combine to make large power plants feasible at a relatively low unit cost for installation. As in the case of reclamation, the future of power development will be greater than its past. It is estimated that 13,000,000 hp. of electrical energy may eventually be generated by water power in the Pacific Northwest, although only 2,000,000 hp. is being made at present.

One of the first commercial hydro-electric plants built in the United States is the one at Oregon City. Those who cross the Cascade Mountains by two of the railways in Washington are lifted up the mountain sides by the force of water harnessed while rushing down those same mountains.

In hydro-electric achievement, as in the case of reclamation, the work of the individual engineer has often been submerged in the activities of a large organization, but the late J. L. Stannard, M. Am. Soc. C.E., should be mentioned whenever municipal power supply is discussed.

In the Pacific Northwest, as elsewhere in the United States, modern highway construction is of recent origin. The topography of the country makes highways very costly, and in view of the relatively sparse population of these states it is all the more surprising to find so large a mileage of improved roads. The locations of some of these highways through the mountains are as spectacular as those of the early railway lines. In fact, the scenic beauty made available by some of them was previously inaccessible except by long and difficult trails. The great Columbia River Highway, which was opened for a hundred miles above Portland in 1915, was the first of its kind in the West. A striking feature of recent highway construction in the Pacific Northwest is the building of many magnificent bridges.

ENGINEERING IN INDUSTRY

About 50 per cent of the entire business of the Northwest coast country consists of lumbering and allied activities. The size of the trees and the precipitous slopes in much of the large forest area, have resulted in the development in this country of lumbering methods which are unique and unprecedented in scope. Engineering has been applied to industry here in saw mills and in the woods, where the logging of trees 6 and 8 ft. in diameter on the steep mountain sides is done entirely by machinery.

At present the most modern method of handling the large timber west of the Cascade Mountains—where the logs are of enormous size and very heavy, and the ground slopes are very precipitous—makes use of the sky-line engine, or combination skidder and loader. The sky-line system has both a head and a tail spar tree, with a high cable suspended between them, along which logs are carried entirely free of the ground. These spar trees are placed as far as 3,000 ft. apart. Much less railway construction is required in such logging than in former methods, and some of the country is so rugged as to preclude the building of railways or the use of surface methods.

Since practically all the settlement in the Pacific Northwest has taken place since 1880, it is obvious that most of the growth of the region lies ahead rather than behind. It would be difficult to find a country that is more nearly self-sufficient or one that enjoys a better climate. Its scenic grandeur and beauty are recognized as being unsurpassed anywhere in the world. The engineer's part in making this region enjoyable for its 3,000,000 inhabitants, as well as for its thousands of annual visitors, and in the creating and fostering of industry and commerce, has been outstanding, both by reason of the opportunities and problems which are here presented, and because of the creditable way in which they have been handled.

The shores of this newest of lands are washed by the waters which also touch the oldest civilized countries—China, India, and Japan. These are the most populous countries on the earth, and the impetus to trade which will come from an improved status of the masses in China, India, and Russia will be felt first in the regions adjacent to the Pacific. It is 4,300 miles from Puget Sound to Yokohama, but distance now is measured in time and cost of transportation, not in miles. Already there are ships which cover that distance in 9 days, so it does not require much imagination to think of the journey's being made in 7 days—only a week from the Pacific Northwest to the Orient, where dwell 500,000,000 people!

The work of the engineer in the Pacific Northwest will continue to be of relatively great importance in planning and building for the larger sphere which this region will occupy in the world's activities.

The Columbia River—for Irrigation and Power

THROUGHOUT the country, and particularly in the West, engineers have shown great interest in the hydraulic possibilities of the Columbia Basin. This interest found expression at the Tacoma Convention of the Society in the papers by Major Butler and R. K. Tiffany, presented before the Technical Meeting on July 8, and here abstracted.

In connection with the nation-wide investigations undertaken by the Corps of Engineers in the interests of navigation, power, and flood control, a comprehensive survey of the Columbia River between the Canadian border and the mouth of the Snake River will soon be completed. Although the report and its conclusions cannot be made public until released by Congress, Major Butler has outlined the scope and purposes of the investigation to show the possibilities for development of the middle reaches of this great river.

A study of the best uses of the Columbia River must lay considerable emphasis on the subject of irrigation. Some of the basic facts which indicate the need for a sound and forward-looking reclamation program are pointed out by Mr. Tiffany.

Comprehensive Study by Army Engineers

By John S. Butler

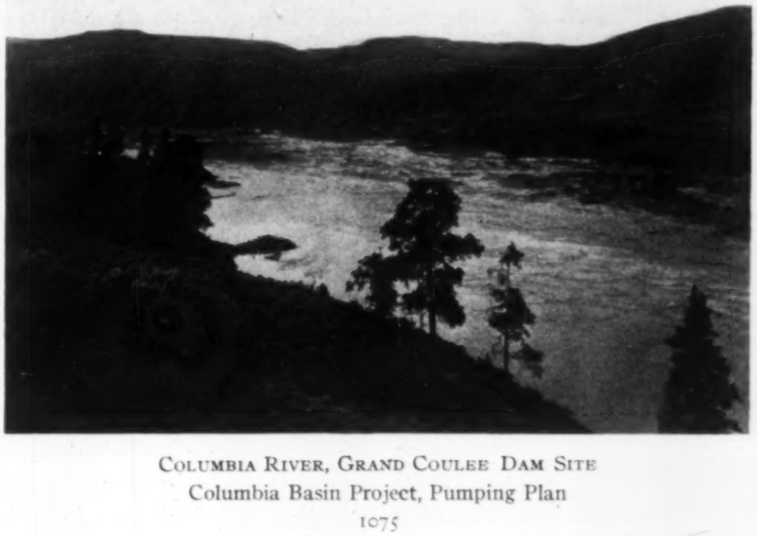
Member American Society of Civil Engineers

Major, Corps of Engineers, U.S. Army, District Engineer, Seattle, Wash.

IT IS the desire and purpose of the Congress of the United States to determine, through the War Department and the Corps of Engineers, U.S. Army, the possibilities of the Columbia River with respect to navigation, power, flood control, and irrigation. This statement applies with equal force to practically all the navigable streams of the United States and their tributaries wherever power development appears feasible and practicable. Nation-wide surveys were finally authorized under Section 1 of the River and Harbor Act of January 21, 1927, and funds for carrying on the work were provided by Congress in lump sums, to be allotted by the Chief of Engineers to the several district engineers as needed.

It may be asked why the U.S. Engineer Department should report on projects having to do with irrigation when there is another bureau of the Federal Government, the Bureau of Reclamation, directly concerned with this subject. Needless to say, the War Department does not undertake to report on streams where irrigation alone is involved, but, in the case of the streams under consideration, there are many interrelated questions that affect the general problem of the full utilization of water resources. Some of these problems are: storage of water in the interests of power, and the resulting effect on flood control, navigation, and irrigation; storage primarily in the interests of irrigation, which may be said to have prior rights over other uses, and its effect on power, flood control, and navigation; and finally, navigability and many other related questions.

It therefore seems necessary and desirable that one single agency of the Federal Government, the one most concerned, should be called upon to prepare a comprehensive plan, keeping in mind the requirements for these four major elements. The two departments of the Government immediately concerned have been directed by higher authority to coordinate their activities in such manner as to avoid a duplication of effort. It is understood that the War Department will submit to the Bureau of Reclamation, for review, that portion of the report which refers to irrigation.



Columbia River, Grand Coulee Dam Site
Columbia Basin Project, Pumping Plan

The Columbia is one of the largest rivers in the United States, and since its source is in British Columbia, it attains an international aspect. The river, which is over 1,200 miles in length, is conveniently divided into three sections by the Canadian border and by the mouth of the Snake River, as may be seen in Fig. 1. The upper section is about 465 miles in length and lies wholly within

British Columbia. The middle section, from the Canadian border to the mouth of the Snake, is 424 miles long, and it is with this section and its tributaries that this article is particularly concerned. The lower section, from the mouth of the Snake River to the Pacific Ocean, a distance of 324 miles, is within the Portland Engineer District.

The drainage area of the Columbia River, in square miles, and its general distribution is as follows:

Section	Drainage Area in Sq. Miles	
Columbia River above mouth of Snake:		
In British Columbia	39,000	
In the United States	64,000	103,000
Snake River		109,500
Columbia River below the Snake		46,500
Total		259,000

The Columbia River has its source at elevation 2,650, in Columbia Lake, British Columbia. Below the Canadian border, between the Spokane River and the Snake River, the Columbia receives from the north and west several swift tributary streams, but from the south and east no stream of importance flows in. Within this section the annual run-off is practically zero for over 9,000 sq. miles of drainage area. This great treeless plateau is one of the most arid regions of the Columbia Basin, the rainfall being only 6 in. at the southern limit near Pasco and about 16 in. in the vicinity of Spokane.

Among the more important tributaries of the upper Columbia River is the Kootenay River, which drains 19,450 sq. miles of mountainous country, of which 14,550 sq. miles are in British Columbia, 3,825 in Montana, and 1,075 in Idaho. The valley of the Kootenay is generally narrow and flanked by mountain ranges rising to altitudes of from 5,000 to 7,000 ft. Below Bonners Ferry the valley is from 2 to 3 miles in width and is very rich in agricultural value. This part of the valley is divided into drainage districts and most of the acreage is under cultivation. The Kootenay has a mean annual discharge of 34,000 sec-ft. Between Kootenay Lake and the Columbia, the Kootenay River falls 330 ft.; and this stretch, with upper and lower Bonnington Falls partially developed, constitutes one of the best sources of power in British Columbia. Regulation of the Kootenay for power purposes in Canada will naturally affect the Columbia River in the United States.

Clark Fork River is the Columbia's largest tributary on the north. Long stretches of this stream, and the lakes tributary to it, are navigable or susceptible of easy improvement in the interests of navigation. Other sections, where the slopes are steep, lend themselves readily to the development of power, to some extent to the control of floods, and to the full needs of irrigation. This last item is of particular importance because of the requirements of the proposed Columbia Basin Irrigation Project, one plan of which contemplates the diversion of the necessary water supply from the Clark Fork at Albany Falls. Many lakes in the upper reaches are fed largely by melting snows and offer a wonderful natural regulation, which can be materially augmented by additional storage at a number of sites, the most important of which are Hungry Horse, Flat Head Lake, Pend Oreille Lake, and Priest Lake. Clark Fork contributes to the Columbia a mean annual flow of about 25,000 sec-ft. The drainage basin of the Clark Fork and its tributaries covers an area of 25,800 sq. miles, of which 1,190 are in Canada.

Spokane River is important because of its developed and potential power sites, provided by numerous falls and the regulating effect of Lake Coeur d'Alene. The river falls over 1,000 ft. in about 100 miles, has a drainage area of about 6,000 sq. miles, and contributes about 8,000 sec-ft. mean annual discharge to the flow of the Columbia. The Wenatchee and Yakima rivers are important in furnishing a much needed supply of water for important irrigation developments. Snake River, the largest tributary of the Columbia, will not be considered here, as it affects only remotely the questions of immediate concern.

The mean annual flow of the Columbia at The Dalles, during the period from 1878 to 1928, was equivalent to $11^3/_4$ in. over the entire drainage area, or about 146 million acre-ft. The lowest year of record, 1927, produced 58 per cent of the mean, or 85.5 million acre-ft. The highest year of record, 1894, was 154 per cent, or 225 million acre-ft. The lowest and highest rates of flow have been estimated at 41,500 sec-ft. for December 17, 1919, and 1,170,000 sec-ft. for June 7, 1894, respectively. The average annual flow of the upper Columbia at the mouth of the Snake River is 127,000 sec-ft., and at The Dalles, 208,000 sec-ft. Low water in the lower Columbia usually occurs early in the winter, and high water between May 15 and June 15.

GEOLOGY OF THE COLUMBIA RIVER

The following quotation is from an outline of the geology of the Columbia River prepared by Dean Henry Landes of the University of Washington:

> The Columbia River, in geological time, was probably in existence as early as the period of the great lava flows which cover all of southeastern Washington.... As the lava floods rose higher,... the Columbia was pushed farther and farther out of its original course. Today the river... reaches the margin of the lava basin about the mouth of Spokane River. From this point, first to the west and then to the south, the Columbia skirts the basin, with cliffs of basalt along its left bank and walls of older rock upon its right side. The river does not enter the lava basin until about Malaga, a small town below Wenatchee.
>
> At Rock Island Rapids the river has entrenched itself in a series of channels in a large outcropping of basalt or lava. The stream continues to flow upon basalt practically all the way to the sea....
>
> For almost the whole distance from the Canadian line to the mouth of the Snake the Columbia has excavated a great canyon. The walls vary in height from a few hundred feet to three thousand feet or more. The banks seldom rise in vertical cliffs of any great magnitude, and in this respect the canyon of the Columbia is in sharp contrast with that of the Colorado or the Yellowstone.... The rise of the bedrock was no more rapid than the cutting power of the great river and it kept to its original gradient while the adjacent mountains grew in height....
>
> An examination of the Columbia shows that today the bedrock floor of the stream is almost everywhere covered with debris of a varying depth. At only a few places, such as Priest Rapids, Rock Island Rapids, Kettle Falls, and Little Dalles is there solid rock continuously under the river from bank to bank. At other places the low falls or rapids are due to boulders in the channel and not to outcrops of rock in place. . . .
>
> The major portion of the debris which is now so evident within the Columbia Canyon is that due to glacial action.

The unusual climatic conditions of the Northwest are largely responsible for the exceptional water resources of the Columbia and its principal tributaries. The heavy precipitation in the high altitudes of the upper reaches insures ample water to meet all possible irrigation requirements and furnishes an abundant and well regulated supply for electric power, estimated to be about one-sixth of the potential power of the Nation.

FLOW DATA PAINSTAKINGLY ASSEMBLED

Stream flow data necessary for these studies were obtained by the U.S. Geological Survey, Water Resources Branch. The many possible combinations of regulation made these studies long and difficult. They covered 18 years of record at 25 stations, with a storage analysis for 16 reservoirs having an aggregate capacity of nearly 14 million acre-ft.

In these studies, regulation was considered in the interests of power as well as irrigation, and full allowance was made for the future requirements of water for irrigation in British Columbia, northern Idaho, Montana, and Washington, and also for the irrigable areas in the valley of the Columbia. The War Department's preliminary investigations with reference to the Columbia River and its tributaries above the mouth of the Snake River, point to the following facts, which will be apparent to those who are at all familiar with this subject:

1. Navigation on the Columbia River at the present time is important only on the lower, tidal section, where extensive improvements for navigation have been made and are justified by existing commerce.

2. There is practically no navigation on the upper section of the river, and extensive improvements will be justified, if at all, only in connection with the construction of dams for power purposes.

3. By reason of its steep slopes and large low-water flow, the upper sections of the river have large power possibilities, the development of which is contingent upon finding suitable sites for dams.

4. The control of floods on the main stream is not a serious problem and only the lower section of the river is involved.

5. There is a large amount of irrigable land tributary to the Columbia, of which only a relatively small amount has been irrigated.

6. The Columbia Basin Irrigation Project is of particular importance in studies of the upper Columbia because of the diversion of a large quantity of water, about 15,000

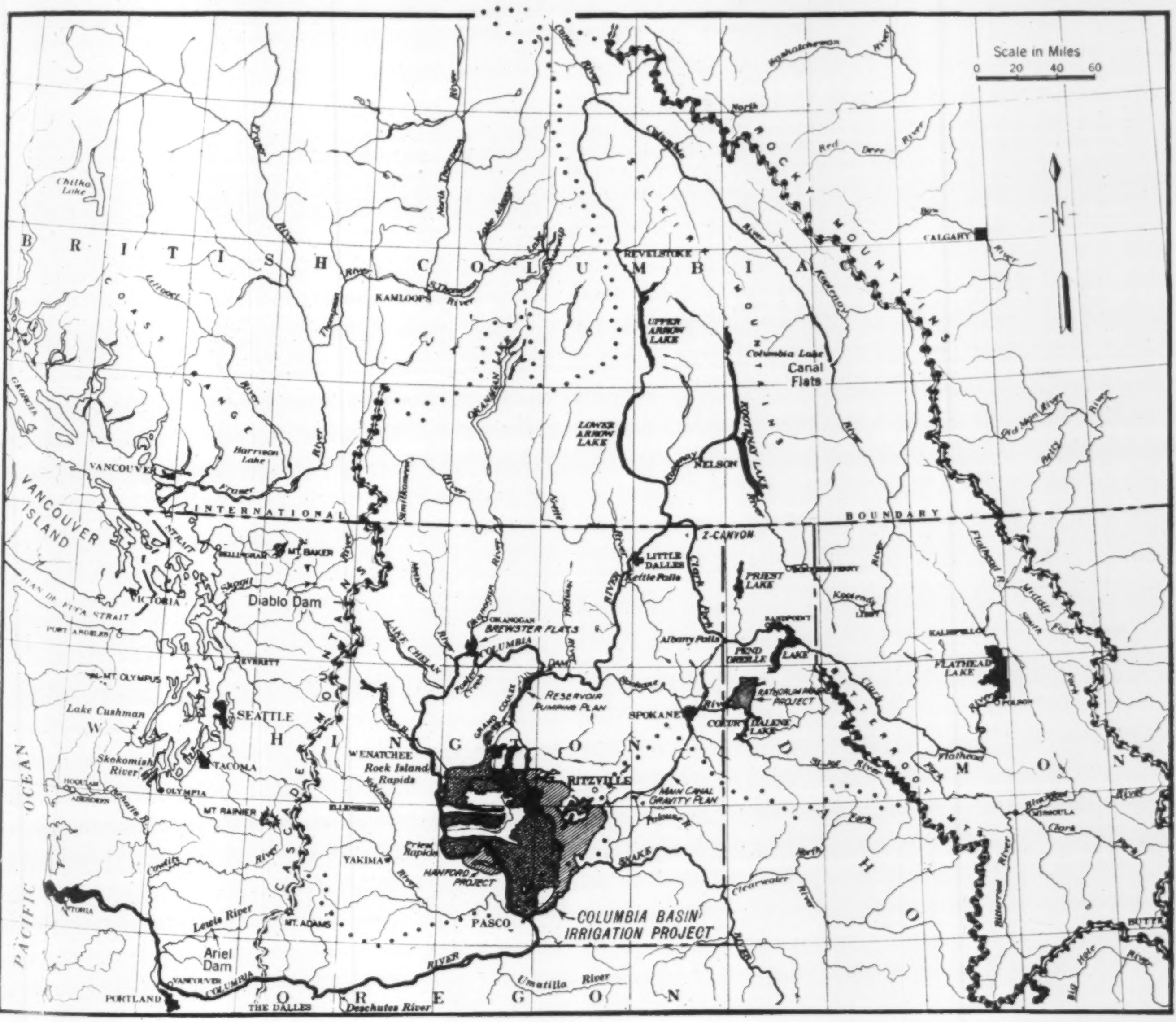

Scale in Miles
0 20 40 60
N
BRITISH COLUMBIA
COAST RANGE
ROCKY MOUNTAINS
SELKIRK MOUNTAINS
Chilko Lake
Lillooet River
Fraser River
Thompson River
North Thompson River
S. Thompson River
KAMLOOPS
Harrison Lake
VANCOUVER
VANCOUVER ISLAND
VICTORIA
STRAIT OF GEORGIA
JUAN DE FUCA STRAIT
PORT ANGELES
MT. OLYMPUS
Lake Cushman
Skokomish River
HOQUIAM
ABERDEEN
OLYMPIA
TACOMA
SEATTLE
EVERETT
BELLINGHAM
MT. BAKER
Diablo Dam
Skagit
MT. RAINIER
Cowlitz
Lewis River
Ariel Dam
VANCOUVER
PORTLAND
THE DALLES
Deschutes River
Umatilla River
COLUMBIA RIVER
MT. ADAMS
YAKIMA
Yakima River
ELLENSBURG
WENATCHEE
Rock Island Rapids
Priest Rapids
HANFORD PROJECT
PASCO
SNAKE
COLUMBIA BASIN IRRIGATION PROJECT
RITZVILLE
MAIN CANAL GRAVITY PLAN
RESERVOIR PUMPING PLAN
GRAND COULEE
Dam
LAKE CHELAN
Okanogan
BREWSTER FLATS
Methow
Similkameen
OKANAGAN LAKE
Kettle River
LITTLE DALLES
Kettle Falls
Z-CANYON
Clark Fork
Albany Falls
PEND OREILLE LAKE
PRIEST LAKE
SANDPOINT
BONNERS FERRY
Kootenai
Spokane River
SPOKANE
COEUR D'ALENE LAKE
RATHDRUM PRAIRIE PROJECT
Palouse R.
Clearwater
St. Joe River
North Fork
WASHINGTON
IDAHO
OREGON
MONTANA
BITTERROOT MOUNTAINS
Bitterroot River
Big Hole River
MISSOULA
Clark Fork
Flathead River
FLATHEAD LAKE
POLSON
KALISPELL
South Fork
Middle Fork
Blackfoot River
BUTTE
REVELSTOKE
UPPER ARROW LAKE
LOWER ARROW LAKE
NELSON
KOOTENAY LAKE
Columbia Lake
Canal Flats
Kootenay River
Columbia River
Canoe River
Bow River
CALGARY
Red Deer River
Saskatchewan River
Oldman River
Belly River
INTERNATIONAL BOUNDARY
PACIFIC OCEAN

Fig. 1. The Columbia River Basin and the Pacific Northwest

sec-ft., which will affect power and to some extent navigation.

As it has been made plain by preliminary study that improvement for navigation can be economically effected only when combined with power development, and as irrigation is recognized as a more beneficial use of water than power generation, it seems evident that the proper approach to the problem for the upper river is to determine the best method of irrigating the 1,500,000 acres in the Columbia Basin Irrigation Project and then make the plans for power development and improvement for navigation conform to this method as far as practicable. This problem was complicated, however, by the fact that two methods of irrigating this land were presented for consideration.

Grand Coulee Reservoir Site
Columbia Basin Project, Pumping Plan

Under the Gravity Plan (Fig. 1), water would be diverted from Clark Fork at Albany Falls and conveyed to the land by gravity through 130 miles of canals, tunnels, and siphons. This plan, which is contingent upon the storage of 1,600,000 acre-ft. of water in Pend Oreille Lake, is further complicated by the fact that it is possible to obtain part of the water from Lake Coeur d'Alene and thus reduce the size and cost of the main supply line between Albany Falls and Spokane. Such a plan would require a modification of laws with reference to the storage and diversion of water in the State of Idaho.

The main gravity canal extending from Albany Falls to the point of distribution near Ritzville, as proposed in previous plans, was about 130 miles long. It consisted of a series of double-bore tunnels 33.5 ft. in diameter, with an aggregate double-bore length of 33.5 miles, the longest section being the 16-mile Bonnie Lake tunnel. Lakes to be created by the construction of dams aggregated 34.5 miles of canal length. Open canals, approximately 70 ft. wide by 20 ft. deep, comprised 61 miles of the total length. Siphons were one mile in length. The estimated cost of the tunnels alone on the main canal line amounted to about $105,000,000. The tunnels and canals would require excavation largely in a hard, compact basalt. The capacity of the canal was approximately 16,000 sec-ft., based on the maximum irrigation requirements for 1,883,000 acres. These data are furnished only for the purpose of showing the relative size of the project.

The Pumping Plan, also shown in Fig. 1, provides for a dam in the Columbia River near the head of Grand Coulee, water to be pumped from the lake thus formed into an artificial reservoir in Grand Coulee. From this reservoir the water would flow by gravity to the project through a main supply canal having a total length of only 9 miles. This Columbia River dam would serve two purposes: it would reduce the pumping lift, and provide head for the generation of primary power for sale and of secondary power for pumping the water for irrigation. The pumping plant would be of unusual size, designed to lift 16,000 sec-ft. of water a maximum height of about 435 ft. from the pool behind the "low" dam, or a maximum height of about 310 ft. if the "high" dam were used. These lifts are only approximate, as they depend upon the stage of the river and the amount of drawdown in the pools.

According to this plan, the bed of the Grand Coulee would be used as a reservoir to receive the pumped water. This coulee is about 30 miles long, from 2 to 6 miles wide, and from 400 to 600 ft. deep. It was scoured out through thick seams of lava flows by the retreating glaciers of the Ice Age. The coulee bed is comparatively flat and is about 600 ft. above the river at low-water stage. The reservoir would be formed by the construction of an earth dam about 60 ft. in height at each end of the coulee. Determination of the water-tightness of this coulee and its suitability for use as a reservoir was a most important problem in connection with the pumping plan. Outstanding engineers and geologists were consulted as to this and many other problems.

The economic feasibility of the Columbia Basin Irrigation Project is dependent, not only on the cost of irrigating the land, but also on the nature and amount of the production from the land; on the markets for that production; on transportation facilities; and on the rate at which settlement would take place, as well as on the type of settler.

POWER POSSIBILITIES MANY

To fully develop the potential water power of the Columbia and to provide for complete canalization, it would be necessary to so locate and construct dams that the backwater from each dam would extend upstream to the next one. To aid in determining the best location of these dams for both naviation and power interests, a survey was made of the entire stretch of river from the Canadian border to the Snake, a distance of 424 miles. Maps were drawn by the aerocartographic method on a scale of 2 in. to the mile, with 20-ft. contours. This survey provided for a line of bench marks from which water-surface elevations were taken at various stages (Fig. 2).

At each of the dam sites on the Columbia, studies were made of the power possibilities. Dams and power houses were designed and estimates of costs were prepared.

The problems of design were difficult because of deep and uncertain foundations and because of the very large volume of flood water, about 1,000,000 sec-ft., for which it was difficult to make suitable provison in the narrow river gorge.

The feasibility of power developments on the Columbia River depends necessarily on the market for the electric power. The amount of such power available for production at any of these Columbia River plants is large, and it will be obliged to compete with present or prospective steam power plants on Puget Sound. Therefore the investigations were extended to include a study of the cost of steam power and the cost of transmitting large amounts of power from prospective plants on the Columbia River to points of dense population near the coast.

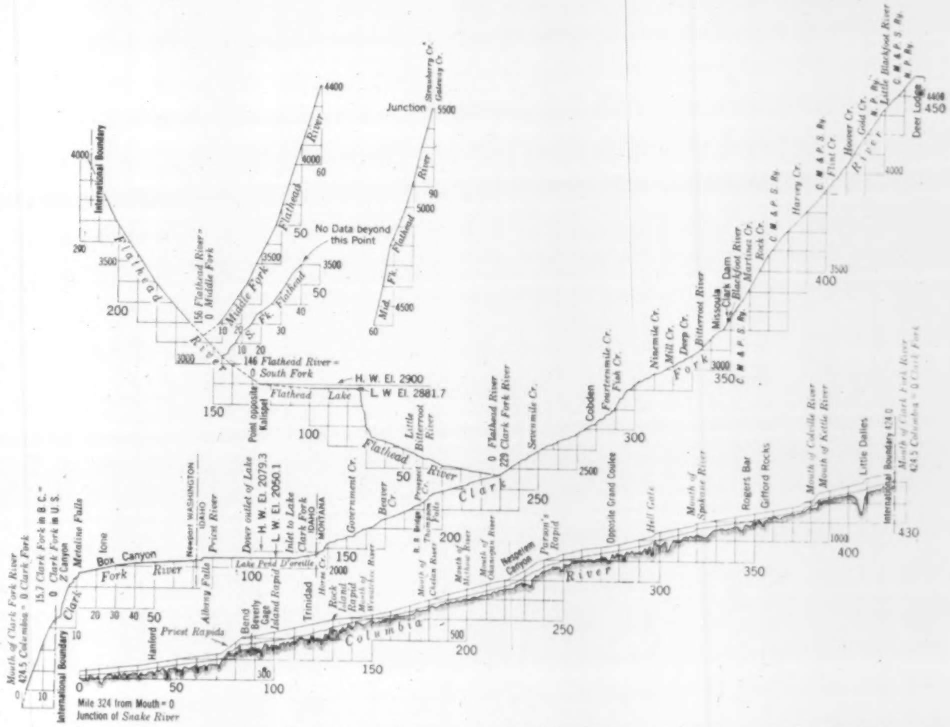

International Boundary
Flathead
Flathead River
Middle Fork
S. Fk. Flathead
156 Flathead River = 0 Middle Fork
146 Flathead River = 0 South Fork
No Data beyond this Point
Junction
Strawberry Cr. Gateway Cr.
Mid. Fk. Flathead River
H. W. El. 2900
L. W. El. 2881.7
Flathead Lake
Point opposite Kalispel
Little Bitterroot River
0 Flathead River 229 Clark Fork River
Sevenmile Cr.
Cobden
Fourteenmile Cr.
Fish Cr.
Ninemile Cr.
Mill Cr.
Deep Cr.
Bitterroot River
Missoula
Clark Dam
Blackfoot River
Martinez Cr.
Rock Cr.
C. M. & P. S. Ry.
Harvey Cr.
Flint Cr.
Hoover Cr.
Gold Cr.
N. P. Ry.
Little Blackfoot River
Deer Lodge
Clark Fork River
Mouth of Clark Fork River 424.5 Columbia = 0 Clark Fork
15.7 Clark Fork in B. C. = 0 Clark Fork in U. S.
Z Canyon
Metaline Falls
Ione
Box Canyon
Newport WASHINGTON IDAHO
Priest River
Albany Falls
Dover outlet of Lake
H. W. El. 2079.3
L. W. El. 2050.1
Inlet to Lake Clark Fork
IDAHO MONTANA
Lake Pend D'oreille
Horse Cr.
Government Cr.
Beaver Cr.
Prospect Cr.
R. R. Bridge
Thompson Falls
Hanford
Priest Rapids
Bend
Beverly Gage
Island Rapid
Trinidad
Rock Island Rapid
Mouth of Wenatchee River
Mouth of Chelan River
Mouth of Methow River
Mouth of Okanogan River
Nespelem Canyon
Parson's Rapid
Opposite Grand Coulee
Hell Gate
Mouth of Spokane River
Rogers Bar
Gifford Rocks
Mouth of Colville River
Mouth of Kettle River
Little Dalles
International Boundary 424.0
Columbia River
Mile 324 from Mouth = 0
Junction of Snake River

FIG. 2. PROFILE OF THE COLUMBIA RIVER AND ITS TRIBUTARIES ABOVE THE SNAKE RIVER

In connection with power plants, the possibilities and costs of improving the river by canalization were given consideration. Such a plan involves the passing of boats over high dams—at Priest Rapids, 90 ft.; at Foster Creek, 160 ft.; and at the Grand Coulee, 220 ft. for the "low" dam, or 350 ft. for the "high" dam. This is an expensive and difficult problem and its economic feasibility is to be proved.

After all data had been collected and designs and estimates of the various elements completed, an effort was made to combine all the best features into a comprehensive plan for the fullest utilization of the natural resources of the region. It is not to be expected that any plan suggested will be accepted in all respects. However, it is hoped that the plans as suggested will be a safe guide for the future consideration of the various elements entering into a comprehensive plan for the fullest and best utilization of the waters of the Columbia River.

NAVIGATION IMPROVEMENTS EXPENSIVE

The following summary of conditions affecting the improvement of the upper Columbia River has been taken from an old report, but still represents conditions as they exist today:

> A pretty thorough knowledge of the river and a careful study of all transportation and economic questions connected with it, convinces me that the river cannot, at least in the near future, become a through highway of commerce upon which will be transported the products from, and the supplies to, the region drained by it. There are too many things against its ever occupying this high position in the world of commerce. These may be summarized as the obstructions, such as rapids and falls of greater or less magnitude, which can only be passed by expensive canals and locks, boatways, or portage roads; the many lesser rapids, where auxiliary power or lining would be necessary; the generally swift currents, requiring a large amount of fuel to surmount, combined with a lack of fuel and the high cost of same; the economic character of the country in the immediate vicinity of the river, which is of such a nature as to furnish or promise but little local business; the topographical character of the river and its immediate vicinity, which renders it difficult to get to and from the fertile regions above it, for it must be remembered that the Columbia flows through a great canyon or depression from 2,000 to 3,000 ft. or more below the general level of the country drained by it; and the dangers which boats must overtake in navigating many portions of its course. The function of the Columbia in this upper portion of its course in the commercial world is and will be, for many years, that certain portions of it will be navigable as feeders to railroads. As such, certain portions will be of great value, and can well receive the attention of the general government.

For such power sites as have been given serious consideration, or for which license has been granted by the Federal Power Commission, provision has been made for the installation of navigation locks at such time as they may be required. The size proposed for these locks is 60 ft. wide, 360 ft. long, and 11 ft. deep on the sills, with lifts ranging from 30 to 50 ft. per lock.

Canalization of the Columbia River would make navigation feasible, but during high water it would still be difficult to ascend the river against the strong currents. The steep slope in the Columbia—about five times that of the Ohio or Tennessee rivers—would make the cost of locks and dams high.

POTENTIAL POWER

The upper Columbia and its more important tributaries have large possibilities for the development of power. The steep slope of the river, averaging about 2.3 ft. per mile for the main stream, together with the large low-water flow, which has natural regulation, make the conditions for power development very favorable. An approximate estimate of the available potential power on the upper and lower Columbia and its principal tributaries within the United States is shown in Table I.

Power plants have been proposed at several sites—at Umatilla Rapids, to operate under a head of from 57 to

29 ft. and generate from 315,000 to 171,000 kw.; at Five Mile Rapids, with a head varying from 105 to 45 ft., to generate from 600,000 to 360,000 kw.; and at the Cascade Rapids, the head to range from 44 to 36 ft., and the power from 315,000 to 150,000 kw.

There are power possibilities on a number of streams tributary to the lower Columbia. The most important tributary, however, is the Deschutes, on which it is estimated that there are about 486,000 kw. of potential power. There are no developed power plants on the main stream of the lower Columbia although there are a number of small plants on tributary streams, aggregating about 50,000 kw. There is now under construction a new power plant on the Lewis River, the installed capacity of which will be 180,000 kw.

TABLE I. AVAILABLE POTENTIAL POWER ON THE COLUMBIA RIVER AND TRIBUTARIES, IN KILOWATTS

	NATURAL FLOW		REGULATED FLOW	
RIVER	90% Time	50% Time	90% Time	50% Time
Columbia River and tributaries in British Columbia (incomplete)	250,000	815,000	—	—
Columbia above the Snake	2,399,000	5,157,000	3,071,000	5,470,000
Tributaries above the Snake	1,616,000	3,138,000	—	—
Columbia below the Snake	1,792,000	3,155,000	2,012,000	2,986,000
Tributaries below the Snake (incomplete)	486,000	—	—	—
Total	6,543,000	12,265,000	5,083,000	8,456,000

DEVELOPED POWER

The four major American public utility power companies operating wholly or in part within the drainage basin of the upper Columbia are the following: the Puget Sound Power and Light Company, a subsidiary of the Stone and Webster Corporation; and three subsidiaries of the American Power and Light Company—the Washington Water Power Company, the Montana Power Company, and the Pacific Power and Light Company. The municipal power plants of the cities of Seattle and Tacoma have an important bearing on the power situation in this section.

Several power developments are proposed or in process of construction in or adjacent to the drainage basin of the upper Columbia River. The Flathead River Development, near Polson, Mont., is located about 5 miles below the outlet from Flathead Lake. Here the Rocky Mountain Power Company plans to build a dam supplying an average head of 185 ft., and according to the Federal Power Commission's license, has three years to complete the installation for 150,000 hp.

A preliminary permit has been granted Hugh L. Cooper of New York City to make an initial installation for 300,000 hp. under an average static head of 245 ft., at Z Canyon on the Clark Fork River. The 1925 application of the Washington Water Power Company for the development of 150,000 continuous hp., under an average static head of 68.9 ft. at Kettle Falls on the Columbia River, is still pending.

The Rock Island Development on the Columbia River was started in 1930 by the Washington Electric Company, and construction is proceeding rapidly on this, the first major power development on the Columbia River.

Beyond question, the outstanding power development on the upper Columbia River is that contemplated at Grand Coulee, known as the Grand Coulee Dam. This development is interesting not only because of the magnitude of the project, but also on account of its relation to the pumping plan of the so-called Columbia River Project. Two heights of dam have been considered, a "low" dam about 220 ft. high above low water, and a "high" dam about 350 ft. above low water. This dam must be designed to take care of a 1,000,000 sec-ft. load flow. The kinetic energy of such a mass of falling water—from 12,000 to 20,000 hp. per lin. ft. of dam—is believed to be without precedent.

The upper Columbia and its main tributaries are not usually subject to destructive floods. The lowlands marginal to Flathead and Pend Oreille lakes, however, are inundated during flood stages, and at infrequent intervals some loss is sustained.

The Kootenay River valley, between Bonners Ferry and Kootenay Lake, is subject to frequent floods. Between Bonners Ferry and Port Hill, on the American side of the boundary, about 30,000 acres of very rich agricultural land have been diked and drained. Probably an equal amount of land can be similarly treated on the British side of the boundary. The dikes are built on the higher land along the river banks, and are endangered during floods. Also, the natural flood storage of the river valley has been materially reduced by the encroachment of the dikes, and the drainage problem for the diked land is made more difficult by backwater from the lake. At low water there is very little fall in the river between Bonners Ferry and the lake.

In addition to the Columbia Basin Project, there are a number of areas of irrigable land which are entitled to full consideration. The Rathdrum Prairie in northern Idaho comprises about 60,000 acres of excellent land, which has been only partially developed. The Hanford Project above Pasco includes over 40,000 acres of fine land suitable for irrigation. There are about 100,000 acres of irrigable land along the Columbia, from the international boundary to the Snake, which are within pumping reach of the river. Of this amount, about 21,000 acres have been irrigated.

For the entire Columbia River Basin, including Snake River, there are approximately 9,000,000 acres of irrigable land, of which amount about 4,000,000 acres have been irrigated.

GOVERNMENT REGULATION ESSENTIAL

The problems to be solved in connection with the preparation of a comprehensive plan of improvement for the Columbia are so many and so far-reaching in their influence that no interest except the Federal Government can arrive at a satisfactory solution. These problems are not only national but have an international aspect on account of the interests of Canada.

President Hoover, when Secretary of Commerce, made an address in Seattle on "A National Policy in Development of Natural Resources." The President is quoted as saying: "We have need that we formulate a new and broad national program for full utilization of our streams, our rivers, and our lakes. We must no longer think in single terms of single power sites, or single storage plants, or single land projects, or single navigation improvements, we must think (and thanks to science and engineering we can think) in terms of the coordinated long-view development of each river system to its maximum utilization."

Irrigation—A National Problem

By Ross K. Tiffany

Member American Society of Civil Engineers
Consulting Engineer, Olympia, Wash.

In connection with this subject of irrigation, the question at once arises, why at this time consider at all any further irrigation development? With considerable surpluses in most lines of agricultural production, and with our agricultural advisers counseling reduction in nearly all the major crops, should production be increased through the development of new irrigated lands?

In the first place, the only parts of our country west of the 100th meridian having 20 in. or more of annual precipitation are small portions of western Oregon and Washington, and possibly other small mountain areas in the Rocky Mountains and California. This is to say that, aside from timbered areas, practically all the land west of the east line of Colorado is either arid or semi-arid, and most of it is fit only for grazing. It is true that there are several millions of acres of so-called dry farm land within these states. But on this land the most that is attempted is to grow a crop of wheat every alternate year. Vast areas once broken and farmed in this way have, because of insufficient precipitation, reverted to desert.

IRRIGABLE ACREAGE LIMITED

The 11 Western states—California, Oregon, Washington, Idaho, Montana, Wyoming, Utah, Colorado, Arizona, Nevada, and New Mexico—contain a gross area of 753,067,000 acres, but a farmed and cropped area of only 44,299,000 acres. Of the total cropped area, some 19,600,000 acres, or nearly one-half, is irrigated land, the rest being mostly dry farmed wheat land and cut-over timber lands.

Increase in cropped area of the non-irrigated land is likely to be very small, if indeed there is any increase at all, for the cost of clearing cut-over land is very heavy, and the good wheat land is all gone. In fact, it is probably a fair assumption that any increase of non-irrigated land will be at least offset by the return to natural conditions of submarginal cut-over or dry farmed lands. Assuming this to be true, the only source of increase in Western agricultural areas lies in further irrigation development.

In this connection, Elwood Mead, M. Am. Soc. C.E., Commissioner of Reclamation, has been quoted to the effect that the maximum of new irrigable land in the Western United States cannot exceed 10,000,000 acres because of limited water supply. Certainly 20,000,000 acres is a very optimistic estimate, especially in view of the steady decrease in precipitation over the greater part of the Western watershed shown by the records of the past forty or fifty years. This means a probable maximum of 65,000,000 acres of cropped land in these 11 states, an acreage less than 10 per cent of their total area.

POPULATION IS INCREASING

For the last thirty years, the increase in population in the five states of Washington, Oregon, Idaho, Montana, and California has been at a rate about $3^1/_2$ times faster than in the United States as a whole, as is shown in Fig. 1. On the basis of past growth it has been estimated that, by 1960, the population of California will probably be from 15,000,000 to 17,000,000, or practically three times what it is at present. It is altogether probable that the population of the other Western states will double within the next thirty years, so that, by 1960, instead of from 10,000,000 to 11,000,000, the 11 Western states previously mentioned will contain from 23,000,000 to 25,000,000.

Flathead River in Process of Development by Rocky Mountain Power Company

Considering the rapid growth of population in these states and the small percentage of their area which can be used for agriculture, even with irrigation, it seems unlikely that over-production will ever become a problem. The best and largest market for staple crops on the present and future acreage of lands under irrigation is and will be within the Western states.

IMPORTANT FACTORS IN WESTERN GROWTH

The rapid increase of urban as compared to rural population, which is clearly seen in Figs. 1 and 2, has been due generally speaking to the development of manufacturing and commerce as compared to farming. It will be worth while to consider some of the leading Western industries with particular reference to the probability of their further expansion and their importance to the Nation as a whole.

For several decades the Northwest has been, and it will no doubt continue to be, the chief source of lumber for the Nation. During recent years, the re-manufacture of lumber on the Pacific Coast has developed a new group of industries employing ever-increasing numbers of men. From 1914 to 1923, the adjusted value of furniture manufactured in the Pacific States increased 317 per cent, while at the same time in the Nation as a whole it increased only 90 per cent. During the same period, the average number of wage earners in the industry increased but 32.5 per cent in the whole United States, while it increased 208 per cent, practically $6\frac{1}{2}$ times as fast, in the Pacific States. The manufacture of plywood, of specially finished and packaged lumber, of cabinet work, and the utilization of mill wastes in the form of hog fuel or for pulp and paper manufacture, all follow the same trend of increasing the number of men employed in the lumber industry.

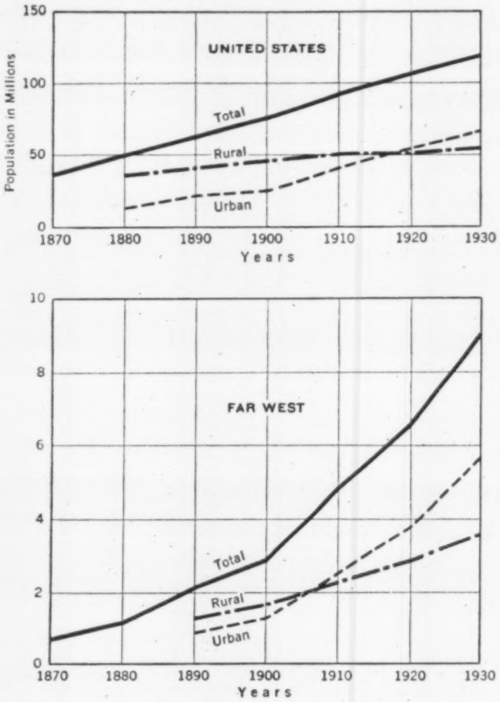


FIG. 1. GROWTH IN POPULATION OF THE UNITED STATES
Compared with That of the Five Western States

Large Eastern operators are now looking to the Northwest for the future development of the pulp industry for two reasons: first, to make sure of a permanent supply of wood at low cost; and second, because electric power, which in a newsprint mill constitutes about one-third of the total operating cost, is abundant and cheap in the Northwest.

Development of mines and mills in the West brought out the need for shops, and from this followed the development of certain phases of the iron and steel industry and of machinery manufacture to very substantial proportions. The paper mills required large quantities of chemicals of various kinds. This called for chemical plants, which are among the latest additions to the industrial development of the area.

RAPID GROWTH OF CANNING INDUSTRY

The growth of the canning industry has been very rapid throughout the whole country of late years. Between 1899 and 1923, the adjusted value of fruits and vegetables canned in the United States increased from about $75,000,000 to over $335,000,000, an increase of more than 343 per cent. During the same period the industry increased 561.3 per cent in the Far West. In 1923 this section canned about 35 per cent of all the canned fruits and vegetables in the United States, and employed nearly one-third of all the workers engaged in the industry.

Another factor which will tend to speed Western development is that of maritime commerce. The next fifty years will in all probability witness a tremendous growth in the Pacific Coast cities. They have water transportation, the shortest route to the Orient, a wealth of natural resources in the hinterland, and, in addition, cheap and abundant hydro-electric power. Pacific Coast commerce has grown rapidly since the beginning of the present century and especially since the opening of the Panama Canal. The rate of growth of foreign trade on the Pacific Coast during the past three decades was more than twice that of the total foreign trade of the United States.

The Nation as a whole has been for many years on an import basis as to beef, mutton, and wool. The country looks to the vast grazing areas of the West to maintain or increase cattle and sheep production, which has not kept pace with the growth of population, as may be seen in Fig. 3. But in order to utilize the range of the mountain and Coast States to its greatest value there must be increased irrigated acreage to provide fall and spring pasturage and feed for the wintering, breeding, and raising of young stock in the lower altitudes and within trailing distance of the range.

There is also a sound reason for the tremendous development of the apple industry along the eastern slope of the Cascade Mountains in Oregon and Washington. The reason is purely economic, and is based on the fact that, by reason of high quality and heavy yields, apples from these localities can be placed in world markets at a cost, including transportation, competitive with the production of any other section. For similar reasons, the citrus fruits, cantaloupes, and head lettuce from California are found in every market in the country. These products are becoming national necessities and cannot be produced elsewhere in the quantity and quality demanded.

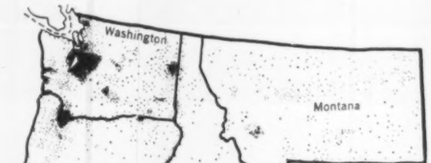


FIG. 2. DISTRIBUTION OF POPULATION IN FIVE WESTERN STATES
One Dot Equals 1,400 Inhabitants

In spite of the quantity of staple crops produced in the West, many products must be imported. As industries expand and population increases, there will be a growing market for foodstuffs. California is, of course, the largest Western market. In 1929, more than 875,000 head of hogs were shipped into California for slaughter. This was 55 per cent of all the hogs slaughtered in that state. Washington the same year received 250,000 head of hogs from outside the state. In addition 12,000,000 lb. of pork and pork products were imported.

IMPORTATION OF FOODSTUFFS UNECONOMIC

Western consumers of pork and of many other farm products are forced to pay Middle Western market prices, plus freight rates from Missouri River centers. This is not economically sound and should not continue indefinitely. Many of the basic materials of industry are produced largely in the West, and to maintain this production economically, there must be in the West a fair proportion of agricultural production.

Besides the present production of important basic raw materials, such as those shown in Fig. 4, the West contains nearly all the vast Federal reserves of timber, coal, phosphates, potash, oil, and gas. In fact, 29.5 per cent of all the land area of the Western states is in permanent Federal reserves, and an additional 25 per cent is owned by the Government, so that, all told, 54.5 per cent of all the land area of these 11 states is owned by the Federal Government. This land is not taxable or otherwise available to help carry the burden of state government or of development and improvement. It is this fact that makes it impossible for the West to finance her own irrigation projects.

REPORT OF CONSERVATION COMMITTEE

The special Committee on the Conservation and Administration of the Public Domain, appointed by President Hoover, considered the factors here outlined and others, and found that the facts warranted a continuation of Federal reclamation. This committee included several Eastern members, who were in the beginning prejudiced against any Federal support for reclamation, but who, after careful consideration, joined in recommending a permanent national reclamation policy.

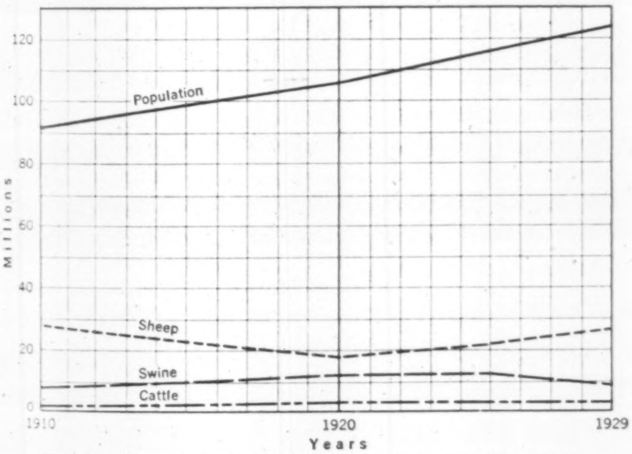


Fig. 3. Comparative Increase in Population and in Food-Producing Animals
In the United States, 1910 to 1929

For the country west of the Rocky Mountains, the possibility of irrigation development is found to lie almost altogether in the three great watersheds of the Columbia, the Sacramento, and the Colorado rivers. The Sacramento lies entirely within the State of California and has already been developed to such a degree that future extensions must depend entirely upon very costly storage. It now appears probable that the state will work out a definite, long-time program for bringing about this development as needed.

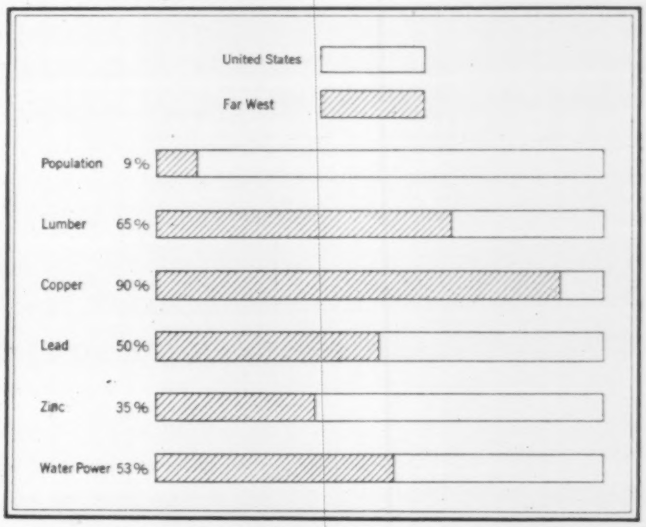


Fig. 4. Percentage of Population and Certain Natural Resources
In the Far West and in the Country as a Whole

On the Colorado and Columbia rivers, however, there are interstate and international problems of considerable difficulty to be solved. The problems of these two streams are essentially different. The Colorado will furnish no more water than will be required eventually to irrigate the land commanded by it, and in the Snake River basin there is sufficient good irrigable land (a probable maximum of four million acres) to require the full diversion and storage possibilities of that stream as now known.

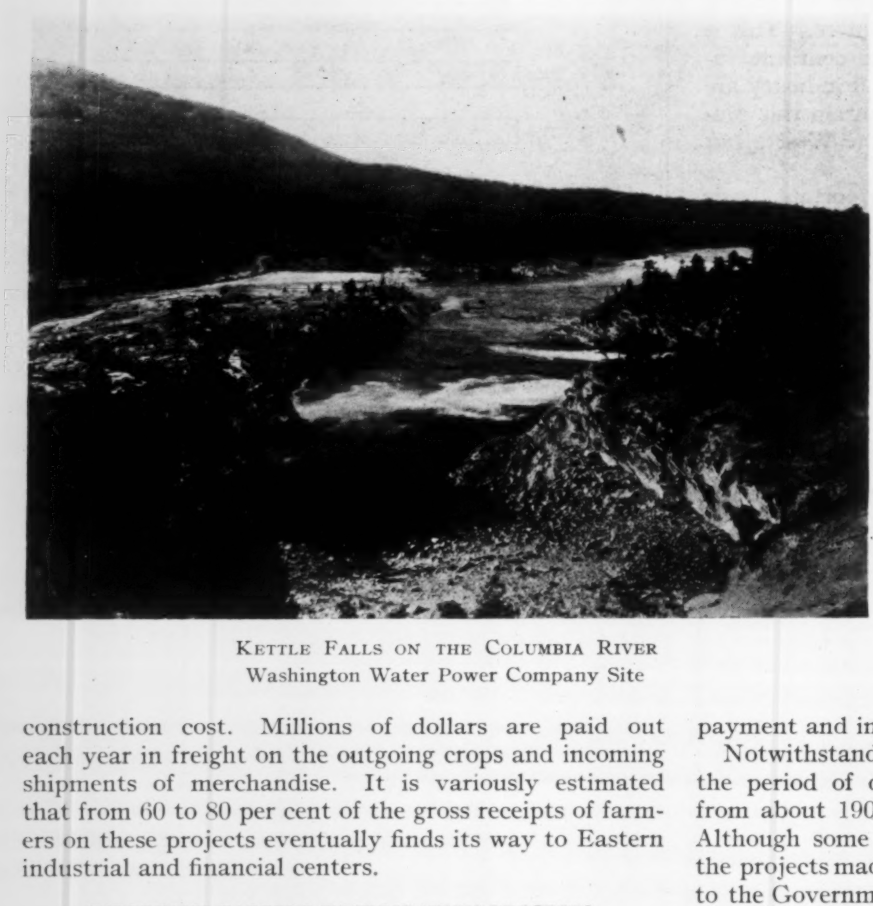
The Columbia, on the other hand, will always send unused to the sea many millions of acre-feet annually. Considering the storage available under the "gravity plan" for irrigating the Columbia Basin Project and allowing for return flow, a minimum supply of 7,000 sec-ft. could be maintained for power below Flat-head Lake, after meeting full irrigation requirements in all the tributary states. Should the "pumping plan" be found more feasible, there is even more surplus above irrigation needs. Storage sites of known feasibility on the Clark Fork have a capacity in excess of 4,500,000 acre-ft.

From the Columbia River and its tributaries there are now irrigated some 475,000 acres in Washington, 247,000 acres in Oregon, 2,060,000 acres in Idaho, and 275,000 acres in Montana. The predominant crops are alfalfa and other forage crops, pasture, sugar beets, and potatoes. The Nation is a heavy importer of sugar, so that the growing of sugar beets supplies a national need. As to the other staple crops, it appears unlikely that Western production will be able to keep pace with Western demand. Wheat on irrigated land is decidedly a minor factor.

In Washington and Idaho the Federal irrigation projects have been uniformly successful and have made a splendid record as to repayment of construction and maintenance costs, as may be seen from the following tabulation:

Project	Construction Cost	Repayment Contracts	Already Paid	
			Amount	Per Cent of Amount Due
Boise	$17,105,874	$14,698,000	$3,765,349	99.5
Minidoka	13,789,934	11,614,416	7,066,490	98.7
Yakima	14,158,897	11,661,747	6,240,333	96.3

On the Government projects in these areas the annual crop production has for many years exceeded the total construction cost. Millions of dollars are paid out each year in freight on the outgoing crops and incoming shipments of merchandise. It is variously estimated that from 60 to 80 per cent of the gross receipts of farmers on these projects eventually finds its way to Eastern industrial and financial centers.



KETTLE FALLS ON THE COLUMBIA RIVER
Washington Water Power Company Site

IMPORTANT FACTORS IN IRRIGATION PROJECTS

Certain elements have always been recognized as essential to the success of an irrigation project. First, of course, come the requirements of good land, not too difficult to till, ample water supply at reasonable cost, and good climate. With the too rapid expansion of irrigation in the early part of the present century, it began to be realized that a market for the products grown is one of the most important factors, and in the last decade land settlement has often been mentioned as one of the most important elements in successful irrigation development.

The former Commissioner of Reclamation of Idaho, George Carter, M. Am. Soc. C.E., states that the average annual rate of increase of irrigated land in Idaho since 1900 has been about 70,000 acres, a remarkable amount for a state having a total population of only 445,000; but markets have been found for the crops, largely in the Pacific Coast States, and there has been at no time any serious land settlement problem in the State of Idaho.

In the State of Washington, the average annual increase in irrigated area from 1900 has been less than 15,000 acres. For ten years, from 1910 to 1920, it ran something in excess of 20,000 acres per year, which was readily absorbed. The Kittitas Division of the Yakima Project, now nearing completion, will bring in 70,000 acres of land, of which some 40,000 to 50,000 acres may be served with water this year. It is already apparent that there will be no land settlement problem here.

There are those who are apprehensive of the success of future projects because of their almost uniformly high unit cost. This is only natural in view of the present agricultural depression, which has brought financial difficulty to so many farmers. It should be remembered, however, that it is not only the irrigation farmers who are in difficulty at the present time, but dry land farmers—and along with them business men of all sizes and varieties.

PROJECT COSTS NOT EXCESSIVE

On the first Government projects in the Snake and Columbia river valleys, the cost of water was from $30 to $60 per acre, payable in ten years, with no interest. This looked so cheap that it encouraged speculation in land, and raw land prices boomed from $2 or $3 per acre to from $50 to $300 per acre. The plan of collection of water charges in vogue at that time placed a premium on speculation, as installments were not collected until the land was actually placed under cultivation. Land was usually purchased by the settler on contract with a small down payment and interest at 8 per cent on deferred balances.

Notwithstanding the high land prices prevailing during the period of development of these projects—that is, from about 1909 to 1915—the development was rapid. Although some individual farmers were squeezed out, the projects made a splendid record of payment of charges to the Government and became soundly established.

From 1913 to 1915 the irrigation district plan for collection of irrigation charges was adopted by the Reclamation Service. Prompt collections eliminated speculation to a considerable degree, so that in the new projects brought in from 1915 to 1920, with water-right costs ranging from $75 to $125 per acre, payable in 20 years, the land prices ranged from $25 to $75 per acre. These projects throughout the Columbia River valley were also rapidly settled and have maintained a fine record as to payment of all irrigation charges.

COST OF FUTURE PROJECTS

The new projects to be built in the future may range in cost from $150 to $200 per acre, but the land will go to the actual settler at from $5 to $25 per acre. There will always be a differential of at least this amount in the value of raw land on account of location, ease of preparation and irrigation, and other factors.

Comparison of past and future costs for land and water rights shows that on the earlier Government projects the annual water-right payments on land costing from $30 to $60 per acre, payable in ten equal instalments, were from $3 to $6. Annual payments on land, excluding the higher priced orchard land, were from $5 to $10 per acre, making a total annual charge, exclusive of operation and maintenance, of from $8 to $16 per acre.

For projects brought in from 1915 to 1920, water charges under the 20-year repayment plan varied from $1.50 to $7.50 per acre, while annual charges on account of land were from $2.50 to $5.00 per acre, making a total of from $4.00 to $12.50. On future projects, assuming that the repayment period will be 40 years, as it has been on all recent projects, the water-right construction charge will be from $4.00 to $6.00 per acre, and the annual charge on account of land, from $0.50 to $2.50 per acre, making a total of from $4.50 to $8.50 per acre.

AN ABUNDANT WATER SUPPLY

In 1925, Congress and the interested states passed laws authorizing allocation between the states of the waters of the Columbia River for irrigation and other beneficial uses. At an early stage in the discussions it became evident that there would be no serious conflict concerning the amount of water to which each state should be entitled, as there is apparently a large surplus above the demands for consumptive use by the upper states.

The Board of Engineers appointed by the Federal Power Commission to report on uses of the upper Columbia River made a very extensive study, and in 1923 filed a report from which the first two conclusions are quoted as follows:

"(*a*) Freedom should be given to fullest irrigation expansion in Montana, Idaho, and Washington, and no rights should be allowed to accrue to the lower interests which would legalize limitation of, or interference with, irrigation above.

"(*b*) The Columbia Basin project is the most important single item to be considered in the uses to be made of Columbia River water above the mouth of the Snake River."

In considering applications for the use of water for power purposes, the Federal Power Commission has indicated that the conclusions of the Board of Engineers' report would be adopted as sound policy.

The only problems of serious difficulty before the Allocation Board are those in connection with overflow rights for storage reservoirs. The legislatures of Idaho and Montana have both enacted legislation aiming to prevent the storage or diversion of water within those states for use outside their boundaries without express legislative approval. Lands around Flathead Lake in Montana, and around Pend Oreille, Coeur d'Alene and Priest lakes, in Idaho, have considerable value for agricultural and recreation purposes, and permanent flooding of them would mean a permanent reduction of production and hence of trade revenues. Some equitable solution of these problems must be reached and no doubt will be reached as the time approaches for utilization of these storage sites.

FEDERAL IRRIGATION POLICY JUSTIFIED

As to irrigation generally, the following conclusions are evident:

1. The development of from 10,000,000 to 20,000,000 acres of land in the West by irrigation will be required within the next 25 to 50 years to supply staple crops for the population of the West; to develop the maximum possibilities for beef, mutton, and wool production for the Nation; and to produce other specialties, such as fruit, long-staple cotton, and sugar beets, for nation-wide use.

Z Canyon of the Clark Fork River
Hugh L. Cooper Company Power Site

2. Future irrigation development is essentially a national problem, because it depends upon streams interstate and international in their courses; because the length of time required for development of most of the remaining projects makes them impracticable as commercial projects; because irrigation must be coordinated with flood control, power development, and navigation in order to bring about maximum utilization, and no feasible plan other than Federal control has been suggested to effect such coordination.

The great addition to the national wealth produced annually upon Federal irrigation projects has justified and will justify the use of national funds and credit in their building. And finally, Federal ownership for benefit of the whole Nation of the most valuable portion of Western natural resources in vast permanent reserves renders it impossible for the states to finance their own irrigation development.

3. Future irrigation development should be based on careful engineering and economic studies, including studies of probable market outlets for anticipated crops, and should be planned and carried on only at a rate

commensurate with the probable growth of market outlets.

FEASIBILITY OF COLUMBIA DEVELOPMENT

As to the Columbia River watershed, the following observations are pertinent:

1. The projects built thus far by the Federal Government in the Columbia River watershed have been notably successful. Charge-offs on account of deficient water supply, poor land, and for other reasons, have been insignificant in proportion to the total acreage reclaimed, and the projects have met promptly the charges assessed against them for construction, operation, and maintenance. These projects produce each year crops greater in value than their total construction cost.

2. There are some 4,000,000 acres in Washington, Oregon, Idaho, and Montana susceptible to irrigation from the Columbia River and its tributaries. Plans should be prepared for the orderly development of all this land, and a development program provided that will bring in from 30,000 to 100,000 acres annually.

3. Irrigation and power development in the Columbia River basin will often conflict, but in some cases will be mutually beneficial. State and Government officials responsible for regulation and distribution of public waters should require that in either of these phases of development due consideration be given to the other phase, to the end that the highest use of each stream be attained. Because power is relatively more plentiful than the water supply available for irrigation, the needs of irrigation should be held paramount.

4. Future utilization of the waters of the Columbia River system involves international problems that will require many years for the collection and study of data and for the negotiation of agreements as to utilization. It is none too soon to begin collecting the essential information and to undertake these studies.

5. The report of the Army engineers, while preliminary in its nature, will cover a comprehensive general study of potential uses of the waters of this stream and its tributaries. It will doubtless outline a general plan for the most complete utilization of the stream. Those responsible for water administration in the various states should supplement the Army studies by detailed investigations of the various possibilities within their respective states. All future development should be required to conform to the general plan, and the states should cooperate to bring about development of the various features as the need for them becomes apparent, through such agencies as will best serve the public interest.

Rock Lake Proposed for Secondary Storage and Canal
Columbia Basin Project, Gravity Plan

Rope Strands for Long Suspension Span

St. Johns Bridge at Portland, Ore., Notable in Design and Execution

By D. B. Steinman

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An era of bridge construction, such as the present, provides outstanding opportunities for the structural engineer. As each new span is built, problems must be solved, and their solution marks further progress in an art already far advanced. The St. Johns Bridge, here described, illustrates how a well conceived design overcomes difficulties by new and better methods. Noteworthy was the attention paid to the esthetic problem, to obtain an artistic structure which would do justice to its scenic setting. A new record for rope-strand cables, both in size and length of span, also resulted, to the great economy of the work. Only a few of the many questions in design and construction which required an answer are covered in this abstract. The complete paper was presented at the meeting of the Structural Division on July 9, 1931, at the Tacoma Convention.

CROSSING the Willamette River near its junction with the Columbia, the St. Johns Bridge, a four-lane highway structure, with a main suspension span of 1,207 ft., is the longest of any type west of Detroit. As to clear height also—205 ft. above navigable water—it sets a high record for required under-clearance. Furthermore, it is the longest rope-strand suspension span in the world, all the eight longer ones being of the conventional parallel-wire construction. These and other distinguishing features brought attendant problems, the solution of which marked noteworthy advances in modern bridge practice.

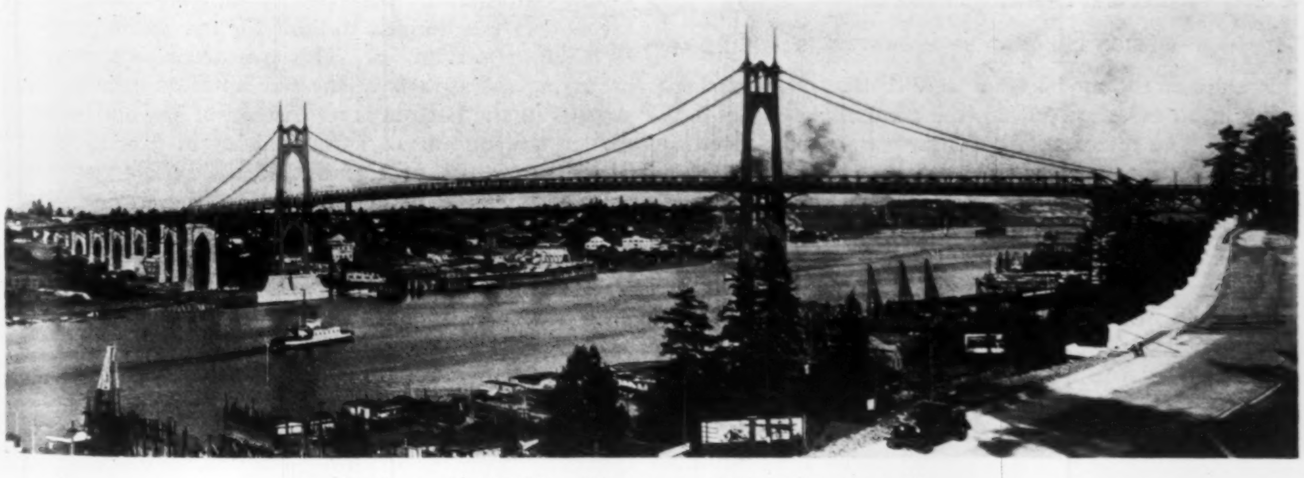
In the design of the St. Johns Bridge, the desire to secure a beautiful public structure was a governing consideration. Framed on one side against a background of forest clad hills, with the spires of evergreens forming the sky line, and on the other, against a panoramic view of city and valley with the snow-clad peaks of Mount St. Helens, Mount Adams, and Mount Hood gleaming above the horizon, the St. Johns Bridge called for a design that would harmonize with its matchless scenic setting.

It was soon found that beauty could be secured without sacrifice of economy. Comparative designs and cost estimates showed that a suspension type, with its naturally graceful cable curves and its other features of superior esthetic distinction, could be built for a cost of $640,000 less than that required for a cantilever bridge of equal capacity.

The design of the lofty main towers is an attempt to secure, in a steel tower, artistic and graceful lines expressing a harmonious combination of beauty and strength. The tower has vertical legs to support the direct load of the cables, in conjunction with batter legs for bracing and stability. With either vertical or batter legs alone, it is difficult to secure a fully satisfying effect. In the one case the tower tends to appear top-heavy; in the other, it assumes awkward angularity. The combination, with its obvious fitness of form to function, is the most satisfying to the eye.

GOTHIC ARCH MOTIF

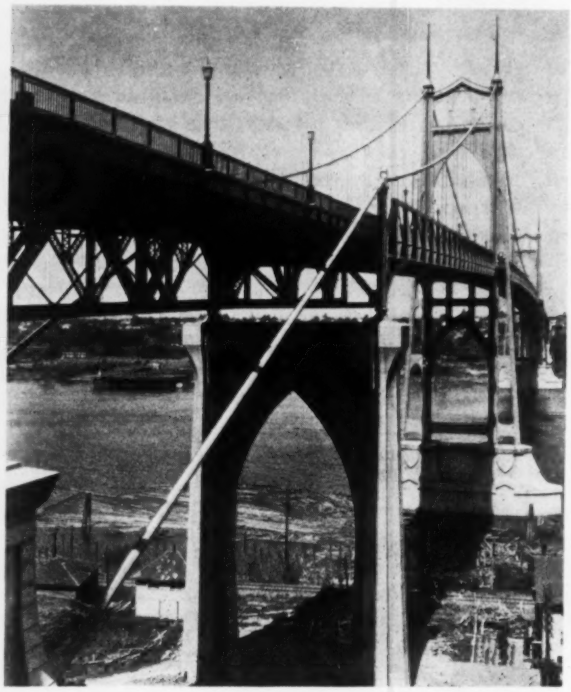
In this design, it was decided to eliminate the stereotyped utilitarian diagonal bracing. Instead, pointed arches both above and below the roadway constitute the dominant motif. Of course, there is the traditional architectural objection. But fortunately we are not bound by medieval tradition when we are endeavoring to develop artistic forms for a new material, and further, the pointed arch seems inherently more appropriate to steel than it is to the voussoir masonry which had to be used before steel became available. In the case of a suspension bridge, the Gothic arch appears to be particularly appropriate as a dominant motif, since its characteristic feature of curves meeting in a point is also the outstanding feature



St. Johns Bridge Over Willamette River, Portland, Ore.
Main Span 1,207 Ft., Clearance 205 Ft.

of the principal lines of the structure formed by the cables.

The architectural theme of the main towers, with pointed arches and buttresses, was continued in the design of the tall reinforced concrete piers of the viaduct approach, so as to produce an effective arcade of vaulted openings. These ascend in height to elevation +160 at Pier No. 10. It is believed that this pier, which is 163 ft. from top to bottom and contains 3,950 cu. yd. of concrete and 150 tons of reinforcing steel, is the largest reinforced-concrete rigid-frame pier yet constructed.



GOTHIC ARCH INCORPORATED IN DESIGN
Cable Bent Makes a Break in the Cable Profile

Special consideration was also given to the final color of the steelwork. Departing from the cold, somber grays and blacks conventionally used in bridge paints, a warmer and more pleasing color was adopted in a light, restful tint of green that harmonizes with the setting while standing out against the dark green of the landscape. The copper-sheathed spires, flashing in the sunlight, are left unpainted so that they will weather to a verde green that will match the tint of the painted steelwork.

The main towers, comprising one of the unusual features of the St. Johns Bridge, are of medium carbon steel, of the semi-flexible type, fixed at the base. Each tower consists of two main vertical columns, 52 ft. from center to center (corresponding to the spacing of the cables), supplemented by outside batter legs spread to 88 ft. $7^{5}/_{8}$ in. from center to center at the base.

Following the mathematical analysis of the deflections and stresses in the main towers, check tests were made on an elastic model. This was over 4 ft. high and was built up of sheets of celluloid to represent the variations of cross section and moments of inertia.

DESIGN CHECKED BY MODEL TESTS

Results of the model tests were distinctly reassuring, being in all cases lower than the computed stresses and strains. The indicated maximum transverse deflection of the tower top, referred to the scale of the actual structure, was $2^{3}/_{8}$ in. from a 30-lb. wind load combined with the effect of the resultant eccentricity of the maximum vertical reactions from cable and truss loads. The maximum indicated fiber stress, after adding the computed stress from unbalanced cable pull, was only 14,000 lb. per sq. in. The model tests showed that the tower was generously proportioned for any combination of stresses to which it might be subjected.

Specifications for the St. Johns Bridge invited bids on twisted-strand construction as an alternative to the parallel-wire cable design. As received, the bids showed a saving of $42,000 by adopting the twisted-strand design. Moreover, the proposal announced an expected saving of two months in the time of completion of the bridge, this saving in time being made possible by dispensing with the construction of foot bridges and by shortening the time required for cable stringing.

In the parallel-wire design, the cable diameter was specified as $12^{3}/_{4}$ in., but the use of twisted-strand construction increased this to $16^{3}/_{4}$ in. An incidental advantage accruing from the increase in cable size was the improved appearance of the bridge as a whole, due to the more substantial aspect of the principal carrying members.

Twisted strands, or "rope strands," should be distinguished from wire ropes. A rope is made up by twisting a number of rope strands around a central strand, and a substantial loss in the magnitude of E results from this additional operation.

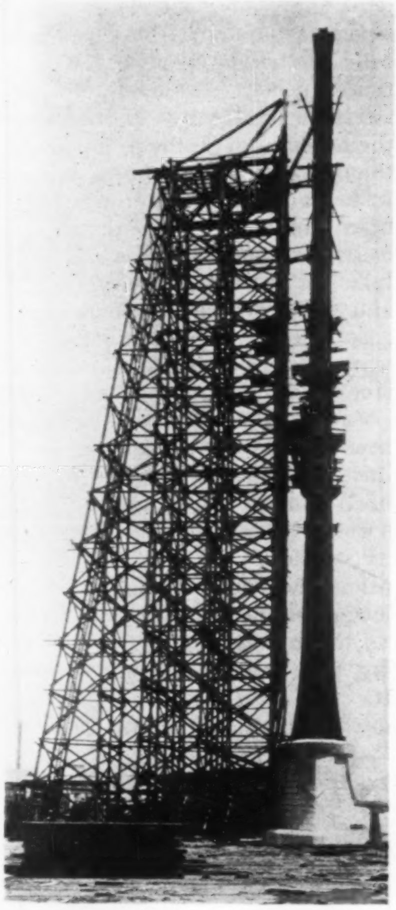
One of the problems in twisted-strand cable construction is the limitation of maximum lateral pressure on the strands, as for instance in the cable saddle, where the bottom strands are subjected to concentrated vertical pressure from the superimposed strands. The unprecedented magnitude of this construction in the St. Johns Bridge made it necessary to give special study to this problem of excessive lateral pressure on the twisted strands.

To aid in a solution of the difficulty, tests were made at the Trenton Plant of the John A. Roebling's Sons Company to determine the effect of transverse pressure on the physical properties of the strands under tension. The tests performed for the St. Johns Bridge showed that even higher pressures did not affect the physical properties of twisted strands, although they did produce visible deformation of section of the wires in the outer layers. It was decided to limit the maximum pressures to 3,300 lb. per lin. in. This was accomplished by rearranging the strands in the saddle, so as to have eight strands in the bottom layer instead of six, and so as to reduce the number of horizontal layers from eleven to ten. In addition, the radius of the saddle curve was increased. As a further precaution, the strands of adjacent horizontal layers were specified to have opposite lay, so as to secure "valley" contact rather than "crossing" contact between the contiguous wires.

ANCHORAGE FOR CABLE-BENT SADDLES

The cable bents are steel rocker bents, used to support the cables at the points where their suspension function terminates and they turn down in steeper lines toward the respective anchorages. Their saddles represent a departure from previous design in that the anchorage for each saddle is provided by means of auxiliary rope strands instead of by friction clamping of the cable strands in the saddle.

Where friction is the sole means for taking up the difference in cable tension on the two sides of the cable-bent saddle, the necessary clamping of the cable in the saddle introduces additional lateral pressure on the cable strands. For the St. Johns Bridge, it was decided to substitute a more positive anchorage between cable and cable-bent saddle, and this was provided by incorporating five additional twisted strands in the cable backstay and anchoring these auxiliary strands to the saddle. The five extra $1^1/_2$-in. strands are pin-connected to the saddle casting at points underneath the main cable bearing and, after leaving the saddle, are incorporated with the other 91 strands to make a 96-strand section for the backstay cable. A tapered metal hood covers the transition section of the cable.



Graceful Main Towers Are of Steel
Timber Falsework for Each Tower Contains 250,000 Board Ft.

A striking departure from the more usual method of tower erection, with vertical creeper traveler, was the adoption of massive timber falsework on the shore side of each main pier, extending to a height of 300 ft. above mean low water. Each falsework tower rested at elevation +30, on a foundation of 56 timber piles, 100 ft. long at the east pier and 50 ft. long at the west pier. There were 252,000 board ft. of timber in each tower, exclusive of the piling.

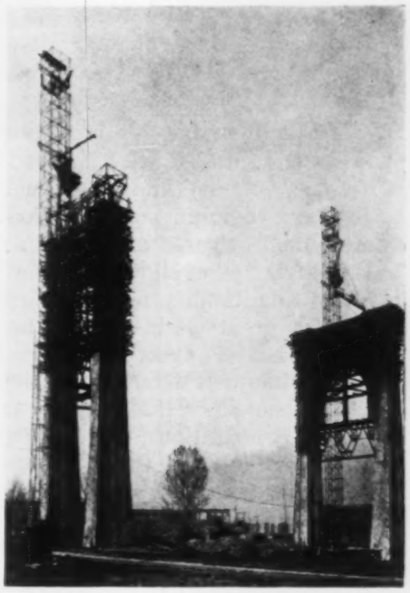
On top of each falsework tower was mounted a stiff-leg derrick with a boom consisting of a stick of Douglas fir 28 in. in diameter and 80 ft. long. This boom controlled the entire erection of the steel tower for its full height, without any shifting of the derrick.

MANUFACTURING ROPE STRANDS INTACT

At the Trenton Plant of the John A. Roebling's Sons Company, each of the 182 rope strands for the St. Johns Bridge cables was manufactured to a length of about 2,750 ft. Each strand was then pre-stressed to a total tension of 150,000 lb. (one-half the ultimate strength) and held at this tension for a half hour. The tension was then reduced to 70,000 lb., corresponding to the average full dead-load stress per strand in the structure.

At this tension of 70,000 lb., the strand was carefully measured to a calculated length, which varied somewhat according to its position in the cable. Corrections for temperature were included in this measurement. The strand was then cut and socketed, wound on a wooden reel, and shipped by steamer to the site. The weight of each strand, not including the weight of the reel, was nearly $6^1/_2$ tons.

The 182 rope strands, each $1^1/_2$ in. in diameter, have a total weight of nearly 1,200 tons. For each cable, 91 twisted strands form a hexagon. This was filled out to a cylindrical section with segmental strips before the outside wrapping was applied.



Pouring the Concrete for the Approach Piers

In the St. Johns Bridge, the hexagonal cable had a major diameter of 11 galvanized rope strands (each $1^1/_2$ in. in diameter), or $16^1/_2$ in. before wrapping. This was given two coats of paint and then the segmented fillers were applied. The fillers consisted of Port Oxford cedar, cut into segmental strips 6 ft. long and treated by immersion for 20 min. in linseed oil at 200 deg. fahr. When practically all the dead load was on the suspended structure, the cables, rounded out with the treated cedar fillers, were given their outside wrapping of No. 9 soft annealed double-galvanized wire.

ASSOCIATED ON THE ST. JOHNS BRIDGE

The bridge was designed by Robinson and Steinman, Consulting Engineers, New York, and was constructed under their direction, with R. Boblow, Jun. Am. Soc. C.E., as Resident Engineer. The work was done under the authority of the Board of County Commissioners of Multnomah County, at a cost of $4,000,000. This board, at the start of the work, consisted of Grant Phegley, Chairman; Clay S. Morse, and Fred W. German. At the completion of the work, the board consisted of Fred W. German, Chairman; Grant Phegley, and Frank L. Shull.

Four successive stages of development have marked the evolution of bridge building—construction, analysis, economics, and esthetics. On the fourth, or esthetic stage, bridge engineers have barely entered. The professional obligation of making bridges esthetically effective, and the possibility of securing beautiful structures in concrete and steel, are more and more being realized. In the case of steel, it is largely a pioneer problem.

Treated Lumber for Irrigation Structures

Pressure Creosoted Material Shows Economy in Western Engineering Practice

By L. Murray Grant

Member American Society of Civil Engineers
Sales Engineer, West Coast Wood Preserving Company, Seattle, Wash

Adaptation of timber to irrigation uses is not new. And yet the treatment of wood—to multiply its resistance to the severe deteriorating influences of climate and to fit it for the special services involved—is comparatively recent. The peculiar problems involved in such treatment were handed over to researchers and Mr. Grant gives some of the answers. As an example of the accommodation of a plentiful raw product to special service conditions, wood treatment as applied to irrigation must rank as notably successful. Some of the problems and the means for overcoming them are graphically explained in this paper, which was delivered before the Irrigation Division of the Society on July 9, 1931, at the Tacoma Convention.

Use of lumber for irrigation structures, such as headworks and flumes, dates back to the very beginnings of irrigation. In nearly all irrigated areas, lumber has possessed the inherent advantages of adaptability and low first cost. Its great disadvantage has been its relatively short life. As all engineers know, it deteriorates very rapidly when exposed alternately to wet and dry conditions, especially in warm climates—and these are the very conditions that apply on most irrigation projects.

Some years ago, while engaged in the manufacture of wood-stave pipe, I became convinced that the use of such pipe on irrigation projects could be very substantially increased if a method could be found to protect the staves from decay during that period of the year when the pipe lines were out of service. Quite naturally, creosoting by the pressure process was first considered, since the value of this treatment had been already demonstrated for other wooden structures. Further, this method was reasonable in cost, and creosoting plants were already established convenient to most centers where wood-stave pipe was being manufactured.

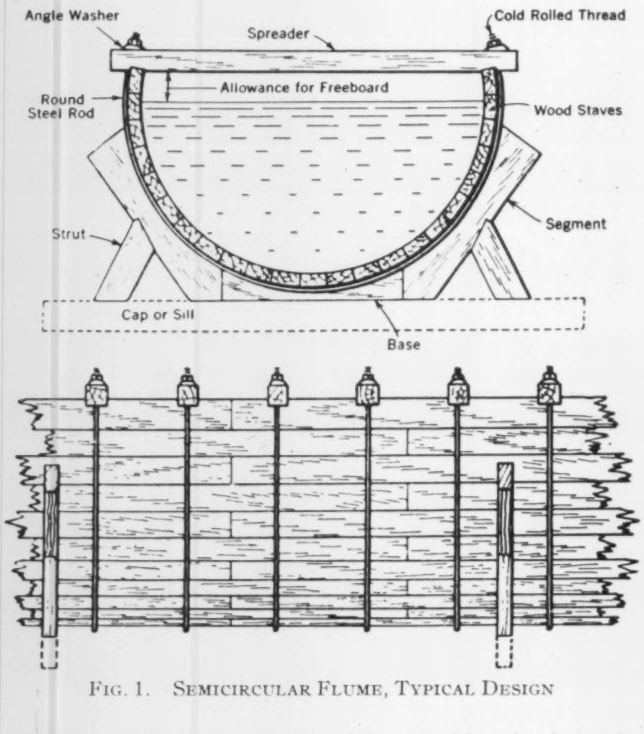
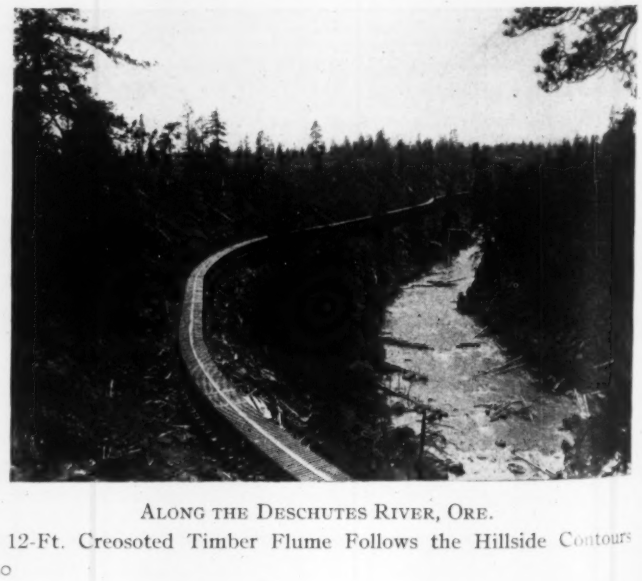


Fig. 1. Semicircular Flume, Typical Design

At that time, 1915, no literature could be found on the use of creosoted lumber for aqueducts on irrigation projects. It therefore became advisable not only to investigate the mechanical problems involved in the use of creosoted staves, but also to learn whether water which had been conveyed some distance in creosoted pipe would have a harmful effect on the germination of seed and the development of plant life. Also it was a question whether such water could be used for drinking by stock and human beings.

After some preliminary work had been carried on, the Bureau of Industrial Research of the University of Washington was induced, in the spring of 1916, to investigate these problems with the cooperation of the West Coast Lumbermen's Association, the Pacific Coast Pipe Company, and the Pacific Creosoting Company of Seattle. It may be of interest to note that this was the first project undertaken by the bureau, which was established in January 1916.

Bulletin No. 1, issued by the bureau in 1917, and entitled *Creosoted Wood-Stave Pipe and Its Effect Upon Water for Domestic and Irrigation Uses*, is the report on this project. One of the conclusions is that, "the amount of creosote that diffuses into water in ordinary creosoted pipes does not have an appreciable injurious effect upon plants either in the time of germination, the percentage of germination, the rapidity of growth, or the general vigor of the plant."



Along the Deschutes River, Ore.
12-Ft. Creosoted Timber Flume Follows the Hillside Contours

During the period of these tests, various methods of treating staves were tried by the creosoting company to ascertain the most satisfactory pipe. As a result, it was found that staves could be successfully treated, after kiln drying and milling to final shape, by the empty-cell process, with initial air, using a final retention of 8 lb. of coal tar creosote per net cubic foot. Plants equipped mechanically and by experience to give this treatment are able to produce a very clean stave, with practically no distortion of section, and hence one that lends itself readily to the manufacture of high-class pipe. Experience with a properly treated pipe or flume has shown that if it is thoroughly flushed when first put into service no noticeable taste is imparted to the water.

Island Siphon Near Harrah, Wash.
Treated Stave Pipe

SEMICIRCULAR FLUME DEVELOPED

When it became evident that creosoted wood-stave pipe could be successfully produced, the advent of the creosoted semicircular flume was not long delayed. I have been unable to ascertain who originated the idea of this flume, but, in any event, it was not used very extensively during the period in which it was built of untreated lumber.

However, it was soon obvious that, when it was constructed of creosoted staves, its usefulness was decidedly enhanced. This is due not only to the long life assured it, but also to the fact that the impregnation of wood with creosote very materially reduces its property of swelling when exposed to water or moisture and of shrinking again when dried. This property has always caused considerable trouble on irrigation structures constructed of wood because of leakage when water is first turned in after a long period of disuse. Not only is a wastage of water involved, but frequently, in the case of flumes, under-scouring of the supports has resulted in the collapse of parts of the structure.

Suspension Bridge, Redmond, Ore.
Crooked River Crossing Requires Reduction in Size from 38 In. to 30 In.

In the semicircular type of flume in which creosoted staves are cinched tightly together by bands passing through headers, the flume is found to be remarkably tight when water is first admitted. Details of design are outside the scope of this paper, but some general idea of current practice can be obtained from Fig. 1 and the accompanying photographs.

Crawling Valley Flume, Canada
Untreated Structure 15 Years Old

Most of the statements here made regarding stave flumes apply with equal force to the various box types when they are well designed and constructed of pressure-treated lumber. To get the full return on an investment in treated lumber, all framing should be done before treatment, and any emergency field cuts or borings should be given the best possible field treatment to prevent the passage of decay spores or fungus to the interior and untreated portion of the wood.

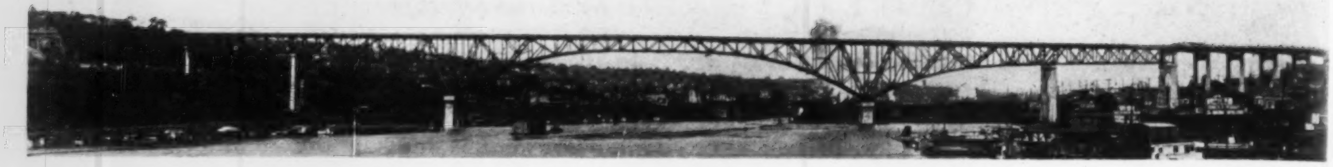
In addition to pipe lines and flumes—which have been discussed first because of the volume of material used in them—there is a decidedly useful field for treated lumber in headworks, diversion structures, and supports for metal flumes of various types. Particularly favorable elements are its low first cost, ease of transportation, and possibility of erection by local labor. It has also been used to some extent, and is deserving of greater use, in bridges over main canals, for cattle guards, and for lateral feeder gates.

ECONOMICAL IN SPITE OF EXTRA COST

Lumber must be shipped into practically all irrigated areas. Its first cost is about doubled by creosoting; but after freight and labor charges are added, which are substantially the same whether it is treated or untreated, the final cost of the treated lumber in place is only from 35 to 50 per cent greater than that of the untreated product. When it is considered that the treated lumber can reasonably be expected to give satisfactory service for from 25 to 35 years, as against from 5 to 10 for the untreated, it is evident that creosoting is a good investment and that the annual cost of treated lumber will compare favorably with that of any other type of construction.

It must be evident that there is a rather wide range of usefulness for treated lumber in connection with irrigation structures; that its use reduces the first cost in most cases, in comparison with any type of construction having a comparable length of life; and that the cost per year, including maintenance, is remarkably low—much lower, in general, than that of other types of construction. The fact that temporary repairs can generally be made with untreated local lumber, in the event of damage from outside causes, is also an advantage not to be passed over lightly.

Acknowledgment is made to the Federal Pipe and Tank Company of Seattle, the Pacific Coast Pipe Company, Ltd., of Vancouver, B.C., and the Canadian Pacific Railway Company, for the use of the illustrations.



DECK-TYPE CANTILEVER STRUCTURE SPANS WESTERN ARM OF LAKE UNION, SEATTLE

Lake Union Bridge Completed

By T. G. McCRORY
MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
CHIEF ENGINEER, DEPARTMENT OF HIGHWAYS, OLYMPIA, WASH.

SEVERAL bodies of water lying to the north of Seattle form a continuous barrier to vehicular traffic, which in the past has been served by four narrow bascule bridges and subjected to frequent interruption by navigation. The six-lane Lake Union Bridge, having a roadway 2,956 ft. long, a main span of 800 ft., and a channel clearance of 135 ft., has been built to remedy this situation. The state, county, and city contributed almost equally to the sum of $2,415,000 required to build it, and its construction is under the direct charge of the State Highway Engineer. The final permit for the bridge was granted by the War Department in September 1929, and two years later, in September 1931, the completed structure will be dedicated. This article is an abstract of the paper presented by Mr. McCrory on July 9 before the Structural Division of the Society at the Tacoma Convention.

TWO natural barriers, Elliot Bay to the west and Lake Washington to the east, make it necessary for through traffic to pass in a north and south direction into and out of Seattle, the chief city of the State of Washington. The Lake Washington Ship Canal, connecting Lake Washington and Puget Sound, creates a further barrier to traffic entering or leaving the city on the north. These conditions and the rapid growth in traffic caused three agencies—the State of Washington, the County of King, and the City of Seattle—to be vitally interested in providing a better rapid transit routing.

The designated routing of the Pacific Highway, the main north and south highway of the Pacific Coast, was formerly over the Fremont Avenue Bridge, a double-leaf bascule having a clear roadway with a curb-to-curb measurement of 40 ft. and carrying two car tracks. However, the State of Washington was interested in establishing a through traffic route which would avoid the delays incident to the opening of a movable bridge.

King County's interest centered in providing a traffic artery for a large and growing section of the country north of Seattle and to a lesser degree for the section lying east of Lake Washington, with Seattle as its trade center. These sections had previously been served by four double-leaf bascule bridges, all of low level, with narrow, inadequate roadways crowded to near their vehicular capacity.

Recognizing the joint interest in a high-level bridge, in 1927 the State Legislature appropriated a sum of $50,000 for necessary surveys and studies to determine the feasibility of constructing such a bridge over the canal. An appropriation of $500,000 was also made for its construction if found feasible. The state made this last appropriation under the provision that the City of Seattle or the County of King, jointly or severally, should during 1927 and 1928 double the amount. The construction of the bridge was to be under the full charge, supervision, and control of the State Highway Engineer. The 1929 session of the State Legislature made an additional appropriation of $1,000,000, which was also contingent on the provision of twice the amount by the City of Seattle or the County of King, jointly or severally. To match both these appropriations the county and the city each provided a like amount.

PRELIMINARY STUDIES

The firm of Jacobs and Ober, Consulting Engineers of Seattle, were retained by the State Highway Department to make the preliminary investigations and to prepare the final plans and specifications for the Lake Union Bridge. Investigations involved the primary consideration of four alternative locations as well as innumerable studies as to such factors as structure types and economic span lengths. The four primary locations, considered in order from east to west, were designated as the Stoneway, Albion Place, Whitman Avenue, and Aurora, these being the names of the streets into which the proposed north approach of the bridge was in each case aligned. Comparative physical data relating to the four routes, based entirely on preliminary studies, are given in Table I.

It will be noted that conditions are progressively more favorable to the west, and these conditions were directly reflected in the estimated costs of the structure. For any type of bridge considered, the cost for the Stoneway location was always greatest and that for the Aurora Avenue location least.

DECK CANTILEVER TYPE ADOPTED

The structure adopted, and now nearing completion, is of the deck cantilever type with an 800-ft. main span, flanking steel spans of varying lengths, and concrete approaches, as shown in the accompanying photographs. The total length of the structure, exclusive of the plaza development at the north end, is 2,956 ft. On the basis of the apparent total cost, there was little difference between the simple span, the cantilever, and the suspension type. For its own economic span length, a slightly lower cost was indicated for the simple-span type, but for a somewhat longer main span, which was desirable if it could be economically attained, the cantilever type was the cheapest. These considerations of cost, added to the fact that foundation conditions, particularly on the south side, were less favorable for a suspension design, caused the cantilever type to be adopted.

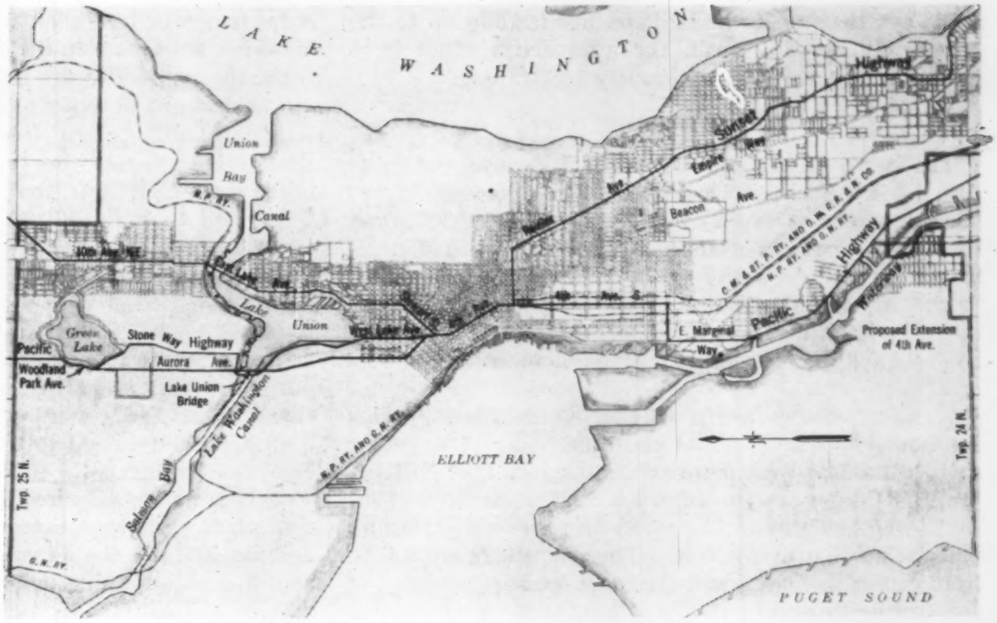


Fig. 1. Map of Seattle

There was the usual contention by some of the shipping interests for high vertical clearances. This was based, in part at least, on the claim that Seattle was a world port; that these waters were open to the commerce of the seas through Salmon Bay Locks; and that the bridge clearance, therefore, should be sufficient to permit the passage, intact, of the higher masted, ocean-going vessels. It was shown that each foot of increased vertical clearance height above 135 ft. meant an increased cost of over $30,000, and that the trivial cost of housing or telescoping the topmast of the occasional vessel, if any, that could not be accommodated by a 135-ft. clearance, was no justification for imposing upon the bridge structure, and upon the highway traffic, an additional investment burden of over a half-million dollars.

Table I. Comparative Physical Data on Four Sites Studied

Item	Aurora Avenue Line E	Whitman Avenue	Albion Place	Stoneway Line B
Route data:				
Total distance between common points, in feet	16,640	17,130	16,660	17,000
Distance across Woodland Park, in feet	2,960	4,055	4,073	4,130
Distance across Lake Union between shore lines, in feet	1,170	1,380	1,520	1,570
Distance across Lake Union between harbor lines, in feet	559	785	912	997
Maximum gradient on route	5.1%	4.0%	4.2%	4.9%
Maximum gradient on bridge	2.5%	2.5%	2.5%	2.5%
Aggregate curvature between common points	98°	——	——	235°
Length of Bridge (in feet):				
Length of main structure	1,650	1,800	1,910	2,020
Length of north approach	1,130	—	2,000	2,350
Length of south approach	590	—	960	775
Total length of bridge	3,390	—	4,870	5,145
Foundation Data (in feet):				
Date of borings	1928	1926	1926	1928
Maximum water depth below mean lake level	42	42	42	43
Thickness of mud stratum in bed of lake	0 to 63	22 to 76	60 to 91	4 to 142
Thickness of strata between bottom of mud and top of compact material	2 to 148	6 to 38	6 to 40	1 to 110
Average depth at harbor lines from mean lake level to compact material	115	106	100	90
Maximum depth from lake level to compact material	186	151	172	238
Average depth from lake level to compact material	109	—	—	139

The initial permit granted in 1929 by the War Department, providing a main span of 628 ft., with a vertical clearance of 152 ft. for a main channel width of 150 ft., and of 142 ft. at the south pierhead line, was modified to provide a clearance of 135 ft. for a main channel width of 150 ft., and a clearance of 73 ft. and 125 ft. at the north and south pierhead lines, respectively. The increased cost of the 628-ft. span, due to the Government's requirement of a high clearance at the south pierhead line, and the fact that better foundation conditions would obtain for the main piers if they were moved shoreward, turned the scale in favor of an 800-ft. main span. As this span length would also afford a more ample horizontal clearance for shipping, a request for Government approval of the change was made and a final Federal permit on that basis was granted in September 1929. Final borings and foundation tests were then made at the pier sites and the final designs for substructure and superstructure were promptly completed.

PLAZA DEVELOPMENT AND CONCRETE APPROACHES

The concrete viaduct of the north approach proper terminates at a point 80 ft. north of the north line of North 36th Street, where the viaduct roadway is approximately 21 ft. above the grade of Aurora Avenue. Beyond this point a plaza development has been provided, with the main roadway intersecting the Aurora Avenue grade at the south line of North 39th Street, 1,016 ft. from the bridgehead. Diagonal roadways extend to Linden Avenue, one block to the west, and to Whitman Avenue, one block to the east.

South of the Lake Union Bridge, the development of Aurora Avenue will consist essentially of constructing a very wide thoroughfare under somewhat rigid topographic restrictions.

On the North side of Lake Union the reinforced concrete approach structure is 695 ft. in length. This is composed of 12 spans ranging in length from 42 ft. to 63 ft., and in height above the lower roadway, from 21 ft. to 112 ft. In the north 42-ft. span, the width of the deck is slightly flared to correspond with the diagonal roadways that are to be built on fills leading up to the structure. On the south, the approach viaduct is of reinforced concrete, approximately 148 ft. long.

STEEL SUPERSTRUCTURE

The steel superstructure consists of a cantilever structure 1,450 ft. in length, with a main span of 800 ft., two flanking spans having a combined length of 425 ft., and three 75-ft. approach spans, giving a total length of about 2,110 ft. The profile of the ground on the center line of the crossing required that the structure throughout its entire length, except for the 42-ft. span at the end of the north approach, be placed on a uniform gradient of 2.5 per cent.

Clearance requirements at the pierhead lines, which are not symmetrical about the center line of the main span, combined with permissible pier locations on land, rendered necessary unbalanced anchor arms for the cantilever structure. The south arm is 300 ft. in length and the north arm is 350 ft. The main piers are 800 ft. from center to center, and the main span is composed of two cantilever arms each 325 ft. long, and an intervening suspended span 150 ft. long. All the panels are 25 ft. long. The trusses over the main piers are 108 ft. 6 in. deep and are spaced 40 ft. between center lines. The structure is of the deck type throughout. On the cantilever structure and flanking spans, the floor beams are composed of plate girders between trusses with solid web brackets outside.

Where the reinforced concrete roadway slab is on steel spans it is generally $6^3/_4$ in. thick, but where the stringers adjacent to the truss chords have a wider spacing, the slab is thickened. The sidewalk slabs are $4^1/_2$ in. thick and the sidewalks have a clear width of 5 ft. As for the concrete approach structures, the width of the roadway between curbs is 57 ft. and the curbs are 16 in. high, with a face batter of $3^1/_2$ in.

Contract for the main superstructure was awarded on July 1, 1930, and work was started immediately. Eleven months later nearly all the structural steel was in place; the closing chords of the suspended span were in position; and the riveting was about 75 per cent completed.

FOUR PRINCIPAL PIERS CONSTRUCTED

Timber bearing piles support the two main (water) piers and the north anchor pier, but the south anchor pier is founded in direct bearing on firm blue clay. For this pier the open excavation method was used without sheet piling or extensive shoring. The work was done during a relatively dry period so that only a small amount of bracing was required to maintain the sides of the excavation.

Preliminary dredging was carried out at the sites of the main piers and the north anchor pier in preparation for starting the cofferdams. At the north anchor pier, which is located almost at the edge of the water, the cofferdam wall was composed of 6 by 12-in. fir timbers with dove-tailed tongue-and-groove splines spiked to them. These sheet piles were driven generally to an elevation well below the base of the piers. For some piles, jetting was resorted to, in order to obtain the necessary penetration. On completion of the pier, the sheet piling, which was left in place, was cut off evenly at an elevation just above the average lake level and the cofferdam was backfilled about the pier with selected materials almost up to the tops of the sheet piles.

For the main pier cofferdams, steel sheet piles were used, of Lackawanna deep arch-web section, 16 by 5 by $^3/_8$ in., weighing 25 lb. per sq. ft. of wall. The sheet piles for the south main pier were ordered 72 ft. long, and for the north main pier, 70 ft. long.

DRIVING THE PILES

Steel sheet piles were driven in pairs by a specially built driving head with a groove to admit the double-pile section used on the hammer. After they were driven as far as practicable, the work of excavation was begun, but the driving progressed well in advance of the excavation. After the cofferdam frames were progressively assembled and settled to their final position, the excavation was completed. At the main and north anchor piers, jetting was required for all the bearing piles. The cofferdam walls were carefully marked for the pile rows and for each pile the jet was accurately located in a horizontal position. The hole was jetted to its full depth; the jet was withdrawn; and the pile was then inserted and driven. Occasionally it was necessary to jet a hole the second time. The driver rig carried the jet pipe and the operating lines. Extra strong 5-in. pipe was used for jet. Water was supplied through 5-in. armored pressure hose, and the jets ranged, in tip-diameter, from $1^1/_4$ to $2^1/_4$ in. inside. The south main pier required the deeper jetting and the longer piles. For this the jet pipe was approximately 175 ft. long.

Generally the piles were driven to a top elevation about 50 ft. below the water surface, and to practical refusal. Sometimes it was not possible to get full penetration because of the jetted hole filling in, or because of the skin friction of the pile in hard material. Pulling of such piles was impracticable. Jetting alongside of a pile was not feasible because of the close pile spacing, the depth of the water, and the depth of penetration.

After the piles for a pier had been driven, load tests were applied to a selected number of them. These tests indicated that some of the piles did not have sufficient penetration to safely carry the required loads. Re-driving tests were then made on piles selected at random. When these tests confirmed the load-test indications, general re-driving was resorted to. Load tests then indicated that the piles had been brought to a proper bearing. Re-driving was necessary on about 25 per cent of the piles to secure the necessary bearing. For this work best results were obtained by the use of a follower working with a hammer in portable leads carried by a derrick boom. The follower was guided onto the piles by a diver. In some locations piles penetrated a relatively great depth in hard clay. It is thought probable that piles driven intensely against entrapped water in the holes created hydraulic pressure around neighboring driven piles, thus impairing their skin-friction resistance, and occasionally raising a pile previously driven to practical refusal.

ACKNOWLEDGMENTS

I am especially indebted to J. Jacobs, R. H. Ober, O. A. Piper, and to R. M. Murray, Members Am. Soc. C.E., for the furnishing of data and for their help in the preparation of this paper.

Problems of Water Measurement

In Gages, Structures, Canals, Culverts, Reservoirs, and Streams

MEASUREMENT of flowing water is one of the oldest of the sciences. The many ramifications of the problems involved are always a challenge to the designer of works for the collection, control, storage, and use of water. At the Tacoma Convention, on July 9, the Irrigation Division was fortunate in hearing Mr. Stevens discuss the important features of our knowledge concerning the use of models for determining flow characteristics through hydraulic structures. He pointed out the importance of understanding the limitations of models and of intelligently analyzing the experimental results.

Before the same Division meeting, Mr. Hall explained how errors creep into readings of rain gages, and of automatic stage recorders for reservoirs and streams, and warned against the inconsistencies that may occur in current meter gagings due to changes in cross section and rating curves.

On the same date, at a joint meeting of the Power and Construction Divisions, Mr. Scobey presented an illustrated talk based on a lifetime devoted to the study of flow phenomena encountered in open conduits. These three papers are here abstracted to give the salient points contained in the original presentations.

Models Predict Flow Through Structures

By J. C. Stevens

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In connection with the theory of relativity, it has been stated that the dimensions of all material things could be increased a hundred times without our being able to detect the difference. One might find himself some morning a giant 580 ft. high, writing on a sheet of paper as big as a city lot, with a pencil 50 ft. long, and feel neither chagrin nor elation.

Instead of simple magnification, we can imagine almost unlimited diminution or grotesque distortion without our having any means of detecting the change. A priori reasoning would therefore lead us to believe that we can make models of any proposed structure or device and accurately determine the behavior of the prototype from the observed behavior of the model. This, however, is only approximately true. We find upon closer scrutiny that a model to a scale of one-tenth its prototype, if built of the same material, is ten times stronger than its original. In order to obtain exact similitude of strength between model and prototype, we must therefore make the model of a substance only one-tenth as strong as that used in the prototype, and equally homogeneous.

HARMONY BETWEEN MODEL AND PROTOTYPE

Certain factors are the same for the model as for its prototype. For example, atmospheric pressure, the accelerating force of gravity, frictional resistances, and the viscosity and density of fluids cannot be arbitrarily reduced to accommodate the experimenter's model. Within certain limits, however, for any factor that remains constant for both model and prototype, a set of equations can be prepared that will represent fairly well the effect of the unreduced factor on the reduced model. In this way, the results secured from models can be transferred to their prototypes and the behavior of those prototypes predicted with a fair degree of accuracy.

Attempts have been made to build models that would be more exact replicas of their originals by choosing for their construction certain materials with properties in some measurable ratio to those of the prototypes. Thus it may be shown that hydraulic similarity will result from the use of mercury flowing through a model whose geometrical dimensions are seven-thirtieths of those of its prototype.

PRINCIPLES OF SIMILITUDE

At the California Institute of Technology, it is proposed to conduct experiments of flow with models of siphons, dams, weirs, and other hydraulic structures in a sealed compartment in which atmospheric pressures are under control and can be reduced to conform to any scale of model. The use of a mixture of water and glycerin has also been proposed in order to secure a fluid the viscosity of which would be completely under control. Celluloid is being used extensively for models of dams, bridges, and other structures. Models of the Stevenson Creek Dam, the Gibson Dam, and recently, of the mammoth Hoover Dam, have been constructed of plaster of paris and diatomaceous earth. Model experiments are now under way at Fort Collins, Colo., on the spillways of the Hoover and Madden dams. Although the prototypes will be of concrete, the models are mostly of wood, and water is used in both.

If actual similitude is desired, the properties of these various construction materials can be accurately determined and compared with those used in the prototype. The scale of the models can then be chosen so as to secure similitude of certain functions, such as deflections, strength, thermal properties, and viscosity.

In practical hydraulics, however, it is seldom necessary to go beyond the ordinarily obtainable materials for the construction of models. Exact similitude is rarely obtainable and it is not at all necessary if the range of the experiments is kept within well defined limits.

This article will not permit of more than a summary of the relationship that must exist between the model and the prototype, and of the limits that must be observed in experiments on models. Let n equal the ratio of corresponding linear dimensions in prototype and model, respectively. That is, the dimensions of the prototype are n times those in the model, which means that the model is built to a $1:n$ scale.

It is obvious that, for geometrical similarity, all homologous linear dimensions must vary directly as n; homologous areas, as n^2; and homologous volumes, as n^3. Among linear dimensions are to be included length, breadth, and thickness; perimeters; radii; velocity; heads due to energy, friction, and pressure; stresses and strains; gradients; and all other quantities that may be represented by a linear dimension.

Kinematical similarity requires that certain relationships exist between time velocity and acceleration. The relation $v = \sqrt{2gh}$ furnishes the key. Since h is a linear dimension, the ratio of linear dimensions varies directly as n. Therefore the ratio of homologous velocities must vary as $\sqrt{n}$; that is, a velocity in the prototype will be $\sqrt{n}$ times that of the corresponding velocity in the model. Now, from $L = vt$, it is evident that, if velocity ratios vary as $\sqrt{n}$, their time ratios must also vary as $\sqrt{n}$. Otherwise linear ratios would not vary as n. If water is flowing at a velocity of 5 ft. per sec. in the model, its corresponding velocity in the prototype will be $5\sqrt{n}$ ft. per sec.

Events that occur in a given time, such as the number of revolutions of pumps and turbines, vary inversely as time, hence their ratios vary as $\frac{1}{\sqrt{n}}$.

NUMERICAL RATIOS OF PROPERTIES

Accelerations are equal in both prototype and model. If a body in the model falls, it will be accelerated just as rapidly as if it belonged to the prototype. The earth's attraction knows nothing of models. Similarly, it may be shown that accelerations from all other forces are equal in all similarly situated structures, regardless of size.

It is obvious also that all constants and ratios; unit weights; coefficients of elasticity, viscosity, and thermal conductivity; specific heat; specific volume; and specific gravity, have the same value in the model as in the prototype, for the same materials similarly situated.

Since force is mass multiplied by acceleration, forces in the prototype are n^3 times those of the model. This applies to pressures, weights, impacts, and reactions. Flow is a product of area and velocity. Homologous areas vary as n^2 and velocities as $\sqrt{n}$. Flow ratios therefore vary as $n^{5/2}$.

Power may be represented as a product of flow and head, whose ratios vary respectively as $n^{5/2}$ and n. Power ratios therefore vary as $n^{7/2}$. Work is the product of force and a length, also the product of power and time, either of which leads to the relation that work ratios vary as n^4. This also applies to energy.

Momentum is a product of mass and velocity. Momentum ratios therefore vary as $n^{7/2}$. A moment, however, is in the same category as work, being a product of force and length. Moment ratios therefore vary as n^4.

It is seen from the foregoing that homologues in model and prototype are expressible as a power function of the scale ratio. That is, for hydraulic similitude, where the model and the prototype have the same values of gravity and density of water, the prototype values must be multiplied by a factor, n^k, in which n is the scale ratio. The exponent k is always a multiple of $^1/_2$ and lies between $-^1/_2$ and $+4$.

FRICTIONAL RESISTANCE

Fluid friction deserves special study. The viscosities of fluids, except as they are affected by temperature, are the same for the model and the prototype. The roughness of surfaces over which flow occurs can seldom be reduced to the scale of the model. In one condition of flow, frictional resistances vary directly as velocity; in another, as the square of velocity; and in still another, according to some intermediate exponent of velocity. These resistances are due to the viscosity of the flowing fluid. No fluid is entirely frictionless. Every fluid will therefore sustain certain shearing stresses, minute though they may be.

If a plate is moved parallel to another plate and the space between them is filled with a fluid, the force required to maintain parallel motion at a fixed distance is a measure of the viscosity of that fluid. When the plates are a unit distance apart, and one is moved, the force per unit area required to maintain unit velocity is the coefficient of viscosity, usually represented by the Greek letter μ.

For freezing water, this coefficient of viscosity is 0.018 dynes per sec. per sq. cm.; for boiling water it is 0.0028, or only about 16 per cent as much. In common parlance, hot water is much more fluid than cold water.

If the coefficient of viscosity of a fluid is divided by its density at the same temperature, the kinematic viscosity (represented by the Greek letter ν) for that fluid is obtained.

CRITICAL VELOCITY

Experiments by Reynolds have shown that, for velocities below certain critical values, frictional resistances vary directly as the velocities. Velocities above those critical values vary as some higher power of the velocity approaching 2. Below this critical velocity, flow is described as stream line, laminar, or filament flow, and it occurs in parallel threads. Above this critical velocity, the filaments break up and the flow becomes more or less sinuous or turbulent.

After the critical velocity is passed, however, an entirely new set of conditions obtains. Stream lines break up and the particles of water, instead of following each other in orderly array, move in sinuous paths from one boundary to the other, or become hopelessly mixed. For complete turbulence, the resistance to flow is proportional to the square of the velocity. Except for very rough conduits or very high velocities, complete turbulence seldom exists. At the boundary, the layer in contact with the periphery does not move at all, and the layer next to this is one of filament flow, so that flow above Reynolds' critical condition is generally a combination of filament flow and turbulent flow. The nearer complete turbulence obtains, the nearer the resistance varies as the square of the velocity. The resistance in turbulent flow is due mainly to the formation of eddies.

There is another critical condition of flow, or velocity, that has been given the name of Unwin's critical velocity. It is the velocity for which the energy head is at a minimum, and it can only exist in open conduits. It is also the velocity of wave propagation and is equal to $\sqrt{gd}$, when d is the mean depth of the flowing prism. For this condition of flow, complete turbulence exists, and the lost head is proportional to v^2. Experiments to determine whether the resistances to flow for velocities greater than Unwin's critical velocity are strictly proportional to v^2 or not, are entirely lacking. This would be a very fruitful line of research.

LIMITS TO BE OBSERVED

It is obvious that either filament flow or turbulent flow must exist in both the model and the prototype. Turbulent flow exists in practically all hydraulic structures. Therefore models should not be made so small that filament flow obtains in them. There is thus seen to be a very definite lower limit to the scale of hydraulic models.

Atmospheric pressures define another limit between the model and the prototype. It has been seen that homologous pressure heads vary as n. In the model, the atmospheric pressure is the same as in the prototype. In the structure, a hydrostatic pressure head of H would in the model be represented by $h = \frac{H}{n}$. If a is the atmospheric pressure, the total pressure head in the prototype is $H \pm a$, and that in the model is $\frac{H}{n} \pm a$. These are evidently not homologous quantities, but experiments may be carried on quite satisfactorily provided that the negative pressure or partial vacuum in the model is not such that, when transferred to the prototype, a so-called complete vacuum is obtained.

A limitation following immediately from the foregoing is that not only must the pressure never be reduced to that corresponding to a complete vacuum in the prototype, but it must never be reduced below that corresponding to the vapor pressure of the water at the temperatures at which the structure is to operate. This limitation is particularly applicable in experiments with model siphons.

PHYSICAL RELATIONS TO BE OBSERVED

In measuring the depths of water over rough beds in models, it is often impossible to determine them with accuracy on account of the shallowness of the water. This limit in model work may frequently be obviated by using a distorted scale. For example, the horizontal dimensions may be m times the vertical dimensions.

If models are constructed to determine the effects of scour or of transporting materials, very great care must be exercised. In 1927, I conducted some experiments to study scour below the then proposed Leaburg Diversion Dam on the Mackenzie River, which are described in the Transactions of the Society, 1929, Vol. 93, page 530. The model was made to a scale of 1:12. About 1,000 lb. of the river-bed materials were mechanically analyzed. Then, by combining certain stock sands and gravels, an artificial composite of material for the model was made up, having grain diameters very approximately one-twelfth those of the river-bed material, and similarly graded.

For small models this cannot be done because, even though it were physically possible to make up a composite material, similarly graded, with diameters $\frac{1}{n}$ those of the natural materials, the extreme fineness would introduce transporting and settling qualities in the model entirely different from those in nature. We can, however, choose any convenient size and grading of materials for the model and—by selecting a proper model scale and by mathematical analysis—develop a set of transfer equations that will be applicable. The limits of both Reynolds' and Unwin's critical velocities must be carefully observed in selecting the scale of the model.

OTHER PHASES OF SIMILARITY

Friction effects between the model and the prototype can never be made similar or completely transferable. An approximate compensation for this disparity in dynamical similarity may be secured by choosing a roughness coefficient for the model that bears a certain relationship to that obtaining in the prototype. This requires that the model scale and material also be properly chosen. For example, very approximate similarity of friction effects will be secured for a rocky channel having a roughness coefficient (n, Kutter or Bazin) of 0.025, by the use of a model built to a scale of 1:50, of smooth concrete with a roughness coefficient of 0.013. This is true provided that such a model channel does not produce velocities equal to or less than Reynolds' critical velocity, or equal to or greater than Unwin's. This fact would be disclosed by a study of slopes and the selection of a model slope that would avoid these limitations.

DISTORTION OF MODEL SCALE

Complete dynamical and geometrical similarity between model and prototype can seldom be secured. We can, however, select a model that will have geometrical similitude and dynamical similitude for one predominating force or factor. This factor can sometimes be investigated alone, and another factor determined from a different sized model. For example, transporting effects might be studied from one model and friction losses from another. Distortion of the scale of a model is generally done at the sacrifice of dynamical similarity. Nevertheless, it may be fully justified in order to avoid other unsurmountable limitations.

The use of models for predicting flow characteristics in hydraulic structures opens up practically a new field of engineering—one in which the practical designer, the trained physicist, and the mathematician must collaborate. Only engineers possessing all these requirements can qualify as research engineers in hydraulics.

Experimentation with hydraulic models is in its infancy. We have barely made a beginning. Yet certain hydraulic phenomena for which a few years ago no explanation could be found have, as a result of research, already become commonplace and thoroughly understood—for example, the critical velocities of Reynolds and Unwin, and the hydraulic jump. There is much yet to be done. We know comparatively little about atmospheric pressures and cavitation effects, transportation of materials, entrained air, eddy losses in expanding boundaries, and frictional resistances for high velocities.

Rarely is the practical engineer qualified, disposed, or enabled to undertake research of this nature. Such work as he has done has frequently been sporadic, incomplete, and miscellaneously directed, and its results have often remained buried.

OPPORTUNITY FOR HYDRAULIC LABORATORIES

Much creditable work has been accomplished at the hydraulic laboratories of the various universities. There has been evidenced by such institutions a very commendable spirit of cooperation. However, the primary function of a university laboratory is student training. Hydraulic laboratories manned by trained experimenters, whose primary objective is research, are greatly needed to solve the problems of the future. The new hydraulic laboratory of the U.S. Bureau of Standards should constitute the beginning of such a program of systematic research into many of the problems with which practical engineers have heretofore been compelled to cope blindly.

Improving the Accuracy of Instruments

By L. STANDISH HALL

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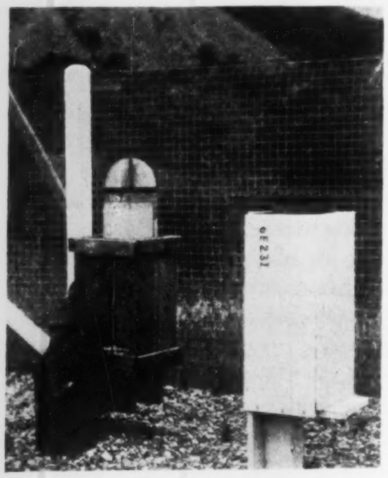
IN THE collection of hydrological data, the engineer is often beset by many difficulties which prevent the securing of accurate results. Many of the sources of error are obvious and can be readily eliminated. Among them are those associated with the human equation, resulting in the incorrect reading of gages or recording instruments, and such obvious mechanical difficulties as the irregular functioning of clocks which have been too long exposed to the elements.

In the gathering of hydrological data, results are sometimes obtained which common sense indicates are not in accordance with conditions as they actually exist. In many instances the discovery of the cause of these inconsistencies is quite baffling, and when the cause has been ascertained the solution of the problem is equally difficult. It may be helpful to discuss some of these common errors and indicate measures for correcting them.

For a long time the unsatisfactory character of the results frequently obtained from the standard rain gage exposed to the wind has been recognized. It is a well known fact that the standard type of cylindrical gage catches less than the actual rainfall, particularly when the instrument is in an exposed location. The inauguration of more intensive studies of the relation between rainfall and run-off, of evaporation (both from soil and open water), of rainfall penetration, and of soil erosion, all require a more accurate determination of the actual quantity of precipitation which occurs.



MODIFIED AND STANDARD RAIN GAGES
San Pablo Reservoir Evaporation and Meteorological Station

In the collection of evaporation data it may often be observed that on rainy days a greater depth of water is removed from the evaporation pan than is caught in the rain gage. This does not indicate negative evaporation, but shows that the evaporation pan, in spite of the evaporation from its surface, is a better rain indicator than the rain gage itself.

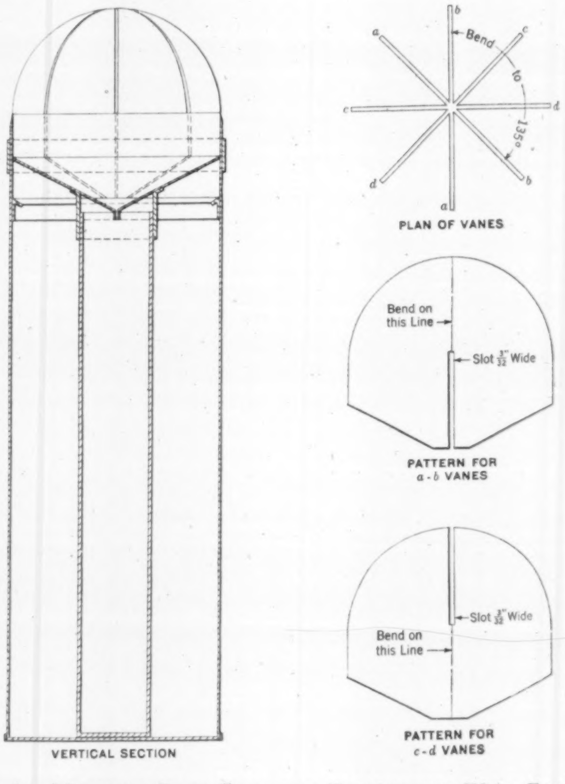


FIG. 1. MODIFIED RAIN GAGE FOR ELIMINATING WIND EFFECT

Having noted this effect at several evaporation stations maintained under my direction, I have developed a simple device which is designed to minimize the effect of air currents near and above the mouth of the receiver, thereby insuring an accurate measurement of the precipitation. The device, which is substantial, compact, and of simple construction, can be easily installed in any standard rain gage.

It consists of a series of fixed and rigid vanes extending above the mouth of the receiver of the rain gage and arranged as shown in Fig. 1. That portion of the vanes which projects above the rim of the receiver is semicircular and has the same radius as the receiver. The lower part of the vanes is shaped to fit the funnel of the receiver, the bottom of the vanes being cut off above the apex of the cone forming the bottom of the funnel, in order to insure the free passage of rainfall into the collector.

As arranged, the vanes have the effect of catching the rainfall on the surface of a hemisphere rather than on the plane surface represented by the mouth of the funnel. In actual operation, rainfall descending at an angle to the vertical strikes against the vertical faces of the vanes and runs into the funnel. If any rain is deflected upward by air currents against the vane, it will drop into the funnel on the lee side of the vane, thus insuring a full catch.

When used in the measurement of snowfall, the receiver and collector inside the gage are removed. A gage of this type, which is shown in a photograph, has been operated for the past year at the evaporation and meteorological station maintained at the San Pablo Reservoir near Oakland, Calif. During this period, the catch of the standard gage was 87.3 per cent of that of the modified gage, or a deficiency of 11.7 per cent. The daily catch of the modified and standard gages as related to the daily wind movement is shown graphically in Fig. 2. The graph indicates that the catch of the modified gage exceeds the standard gage when the total wind movement is greater than 85 miles per day.

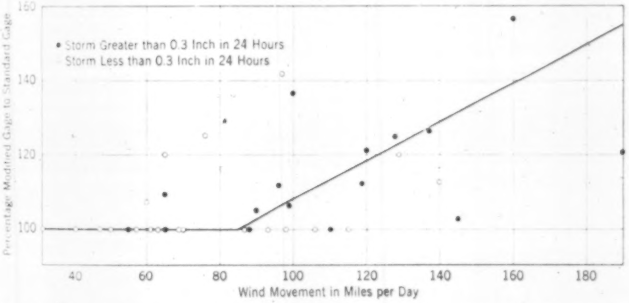


Fig. 2. Relation of Daily Catch of Modified and Standard Rain Gages to Daily Wind Movement

San Pablo Reservoir, April 1930 to March 1931

At the San Pablo Reservoir this modified rain gage has not been operated for a very long period of time, nor has the experiment been of sufficiently extended scope to demonstrate conclusively that this gage actually catches the true quantity of rainfall. However, the results obtained appear to be in the right direction and agree well with those secured by other methods.

ERRORS DUE TO FLOAT CORDS

Early in January 1930, a water-stage register was installed at the Pardee Reservoir near Valley Springs, Calif. The recorder was placed in a sheet-metal house set on top of the parapet wall of the dam at the south end of the gate house. The float well consisted of a 15-in. corrugated iron pipe 200 ft. in length, attached vertically to the upstream face of the dam. It was of sufficient diameter to enclose the float and the counterweight.

Site of Gaging Station on the Mokelumne River

Below Lancha Plana

Four days after the start of the record, a check reading on the staff gage showed that the recorder registered 0.54 ft. higher than the gage. Three days later a similar check disclosed that the recorder registered 0.28 ft. lower than the gage. At this time the length of the float cord between the instrument and the water surface was about 195 ft.

A careful check was then instituted to determine if errors had been made in reading the staff gages or in setting the instrument, and an examination was made of the instrument to determine the possibility of float-cord slip. Then the change in length of a bronze cord, due to the existing changes in temperature, was calculated. None of these examinations offered a clue to the cause of the erratic behavior of the recorder.

On investigation, the float cord was found to be constructed of fine bronze wires twisted into strands in groups of seven, these strands in turn being wrapped around a hemp cord, which formed the core. It was then obvious that the length of the cord was governed by the expansion and contraction of its core. This, being hemp, would be affected by changes in the moisture content of the atmosphere. In maintaining a constant length, the "bronze" cord was of slightly greater value than a piece of hemp string. The type of bronze cord just described is supplied by the majority of manufacturers of water-stage recorders. I have found many engineers inclined to doubt that this cord could affect the registration of a recorder.

METAL BETTER THAN HEMP FOR CORD CENTER

On March 5, 1930, a phosphor-bronze cord with a wire strand core replaced the one originally on the instrument. After this date, the readings of the recorder agreed very closely with the readings of the staff gage. In Fig. 3 is shown the graph of the rise in the Pardee Reservoir between February 27 and March 10, 1930, or for a period of six days before and after the float cords were changed. The distance from the instrument to the water was approximately 100 ft. The value of the wire center is shown conclusively.

If a recorder is operated inside a shelter where humidity conditions are constant at all times, the fluctuations in the length of the hemp core can be practically eliminated. Such conditions may be approximated within a concrete recorder shelter with felt seals at windows and door openings, or in a well pit when the recorder is used to register groundwater fluctuations. However, since this source of error can be so easily eliminated by the use of a phosphor-bronze float cord with a wire core, the use of this type of cord would appear to be indicated for all recorder installations.

Another source of error in the registration of a water-stage recorder is found in cases where the diameter of the float cord varies from that for which the instrument was designed. The same effect may be produced by a mechanical defect in the construction of the instrument. This will cause a deviation in the registration of the recorder proportional to the change in stage.

In installing a water-stage register on a stream, the instrument is usually housed inside a shelter located on the bank. The float well is connected to the stream channel by one or more pipes. Two gages are used, one being a staff or slope gage placed directly in the stream, and the other a staff gage located inside the float well. The recorder is set to the reading of the inside or well gage.

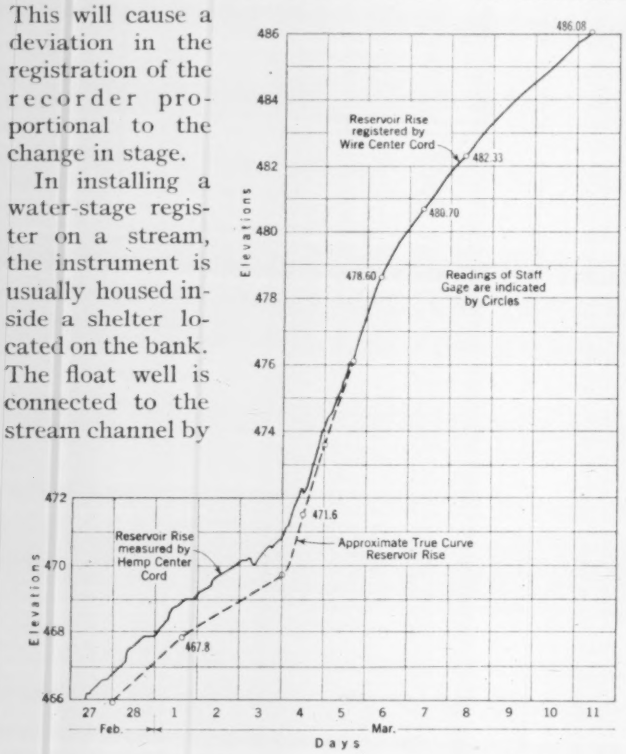


FIG. 3. EFFECT OF FLOAT CORD WITH HEMP CENTER IN MEASUREMENT OF RESERVOIR RISE
Pardee Reservoir, February 27 to March 10, 1930

On many Western streams carrying quantities of sand, difficulty is often experienced in keeping clear the intake pipes between the stream and the float well. A complete stoppage of the intake pipe can usually be readily detected, either from the behavior of the recorder or from a comparison of the inside and outside gages. In some instances, when the shelter is constructed of corrugated iron or wood, water can gain access to the float well by other means than through the intake pipes. This is of no significance when the intake pipe is clear, but if it is obstructed, the water level in the well will fluctuate with the water table. Stoppages of this nature are sometimes difficult to detect, the difference in the reading of the inside and outside gages being ascribed to the settlement of one of the gages.

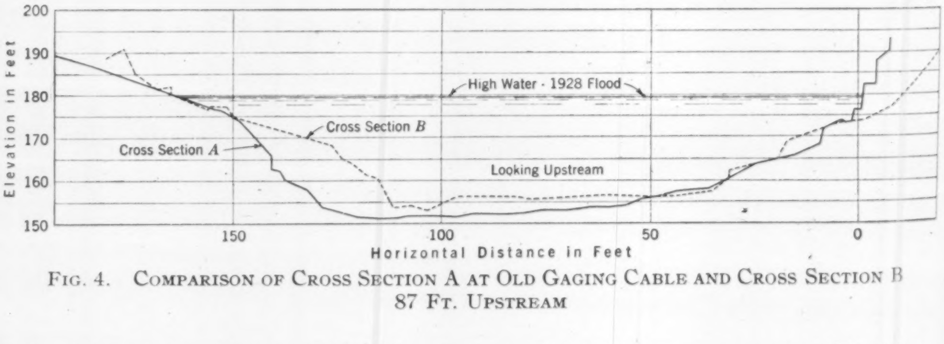


FIG. 4. COMPARISON OF CROSS SECTION A AT OLD GAGING CABLE AND CROSS SECTION B 87 FT. UPSTREAM

Even when the recorder is housed in a concrete well and shelter, a partial stoppage of the intake pipe may occur, resulting in a lag between the river and the recorder well. The fluctuations, in general, may follow those in the river, but on a rising stage the gage in the well will be lower than the river, and on a falling stage, higher. A stoppage of this nature may go undetected for a long period of time.

Interruption or error in the record at various gaging stations caused by the stoppage of intake pipes has led the U.S. Geological Survey to develop a flushing tank for use at stations where this situation has been most troublesome. Such a tank, having a capacity of 30 gal., is placed in the recorder shelter below the instrument shelf. The tank is connected to the lower intake pipe on the inside of the float well. A three-way valve is installed at this point, permitting communication from the river either to the float well or to the flushing tank.

In order to flush the intake pipe, the opening to the float well is closed and the contents of the flushing tank discharged into the river. This method has the advantage of facility of operation at all stages of the river, and is the best solution yet devised for this troublesome problem. In order to insure a continuous and accurate record from water-stage registers, the intake pipe should be flushed at least monthly, on all streams carrying any volume of sediment. It is also desirable to secure levels at each foot mark on both inside and outside gages in order to correct the readings of the outer gage in case of settlement of either. Slope gages are the most troublesome in this regard, because of the possibility of unequal settlement of the ends of the gage.

Although laboratory experiments have been performed to determine the effect of turbulence on the registration of current meters, little has been done to ascertain the actual effect of turbulence under field conditions. Following the flood of March 25, 1928, on the Mokelumne River, the gravel which had formed the bottom under the cable station near Lancha Plana was found to have been scoured out, increasing the area of the section about 600 sq. ft. This cable section was continued in use until June 17, 1930, but it was found that the gagings obtained following the flood were quite erratic. The nature of the control indicates that they should be quite permanent. As shown, the section is formed by the ruins of an old rubble masonry irrigation diversion dam which failed many years ago. The low-water control is formed by the masonry blocks from the ruins lying in the river below the dam, and the high-water control by the remains of the dam itself. The low-water control is subject to a slight shift following very high floods, such as that which occurred in March 1928.

Actual measurements of discharge over a period of 11 weeks showed that deviations from the original rating curve varied from +11 per cent to −12 per cent. In Fig. 4 is indicated the difference during October 1929 in the cross section under the cable, compared with a section taken 87 ft. upstream. Upstream from Section B, the cross section continued with approximately the same dimensions for a distance of 250 ft. Between Sections A and B the cross section expanded as shown. Between the cable and the ruins of the dam the cross section continued as shown in Section A. The rapid expansion terminating under the cable created a turbulent condition, especially near the left bank. Prior to the flood this portion of the section had been occupied by a large gravel bar.

Conditions similar to that indicated in Fig. 4 have been found at many gaging stations. An expansion of cross section occurs to a greater or less degree at all stations taken from the downstream side of bridges. If a bridge having other than a clear span is to be used for gagings, it is preferable to make the gagings from the upstream side, using a boom to keep the meter clear of the bridge piers.

At stations where a cable is to be used, the cost of installation would indicate the economy of expending some funds on surveys of cross sections above and below the proposed site. The condition of the banks can be seen from casual observation, but only soundings will reveal the section below the surface of the water. If a large flood unfavorably changes the bottom or the banks, nothing remains but to move the cable to a new location, if accurate gagings are to be continued. When it is impossible to secure a section free from turbulence, it may be possible to obtain fair gagings by comparing the results of measurements made with screw and with cup-type meters. However, the most certain method of obtaining accurate results is by the selection of a gaging section free from turbulence.

The use of two recorders at a station helps to eliminate or remedy many of the difficulties which have been referred to, including stoppage of clocks, incorrect reading of gages, partial or complete plugging of intake pipes, as well as the separation of the effects of variable surface slopes from shifts in control.

Unusual Flow Phenomena

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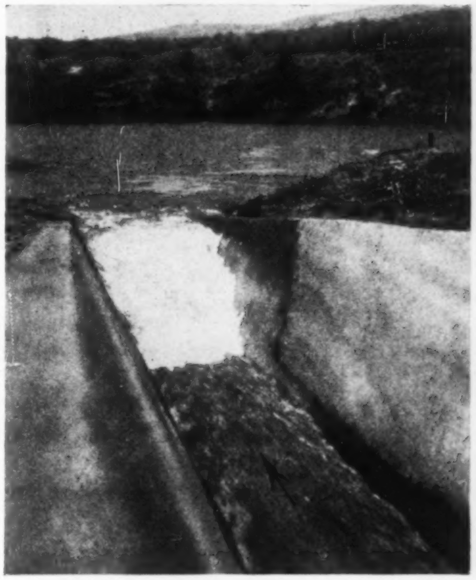
Some of the unusual conditions found in the ordinary commercial operation of hydraulic structures have been entirely neglected in our standard works on hydraulics. While unusual, they are still common enough to deserve attention.

For some 18 years, the Irrigation Division of the new Bureau of Agricultural Engineering and its predecessors in the U.S. Department of Agriculture have studied the flow of water in various types of conduits to determine formulas and conditions that should be considered in the design and operation of such structures in order to convey predetermined quantities of water. Investigations primarily for irrigation have been of equal benefit to hydro-electric and municipal interests. The results of all these studies have been published from time to time as technical papers of the department.

During the field work incident to these investigations, a great many opportunities were offered to photograph unusual flow conditions that have not generally found their way into standard works on hydraulics. Some of them illustrate phenomena touched on in textbooks, but in connection with structures wholly different from those usually associated with these phenomena. Waves of various kinds usually involve complex hydrodynamics, so have been omitted as beyond the scope of elemental books on hydraulics. However, certain types have sufficient bearing on the capacity of structures to make it worth while that a better understanding of them be reached.



Hydraulic Jump Nearly $1\frac{1}{2}$ Ft. High

Jump takes place where reservoir water is held back by water at shooting velocity flowing down the inclined chute. There is no backwater curve and the water impinges into the jump at full velocity.

Our childhood axiom, "Water flows downhill," has prevented many an otherwise good designer from taking full advantage of hydraulics in the recovery of velocity head. What the axiom really should say is that the energy gradient runs downhill. The water surface may—and often does—run uphill. In the case of the hydraulic jump, the surface rises almost vertically.

A much better understanding of the hydraulic jump has developed during the past few years, due to the writings of Kennison, Woodward, Hinds, Stevens, and others. Many examples of this jump, long described as occurring only at the foot of overflow dams, are now found in high-velocity channels.

The hydraulic jump has been utilized as a dissipator of energy. Much of the recent literature emphasizes this property. While there are certain losses in the jump, there is also an excellent opportunity for the recovery of velocity head. It is my belief that the heavy erosion resulting from shooting velocities is reduced in the jump, not so much by the dissipation of energy as by the transformation of kinetic energy to potential energy. Otherwise expressed, part of the velocity head—which is elevation head—may be easily recovered through the jump. The resulting gain in the height of the water surface can be used to command higher ground in an irrigation system, or to provide potential energy for a power plant or a municipal supply.

Velocities in Sinuous Canals

Contrary to general belief, higher velocities follow the inside of the curves in a sharply sinuous canal of trapezoidal section, constructed of material not easily eroded.

Other characteristics of "shooting velocities" (that is, those faster than Unwin's critical velocity) will come to be recognized as definite advantages in planning hydraulic structures. Since shooting velocities have no time for turbulence, the friction losses are not so great as those indicated by flow formulas with ordinary values of friction coefficients.

On the other hand, such velocities easily entrain air to the extent of 30 per cent or more by volume. This fact has generally been neglected in computations. Neither backwater nor drop-down curves exist when a jet at shooting velocities is changed to flow at low velocity or at the sinuous-flow stage. The velocity of propagation that might develop a backwater curve or a drop-down curve is slower than the velocity of water at the "shooting stage."

Backwater and drop-down curves must not be confused with the surface curves developed when water pouring over into a chute accelerates from its critical up to its normal velocity—that is, the velocity attained when the friction losses balance the energy gradient at the same slope as that of the chute.

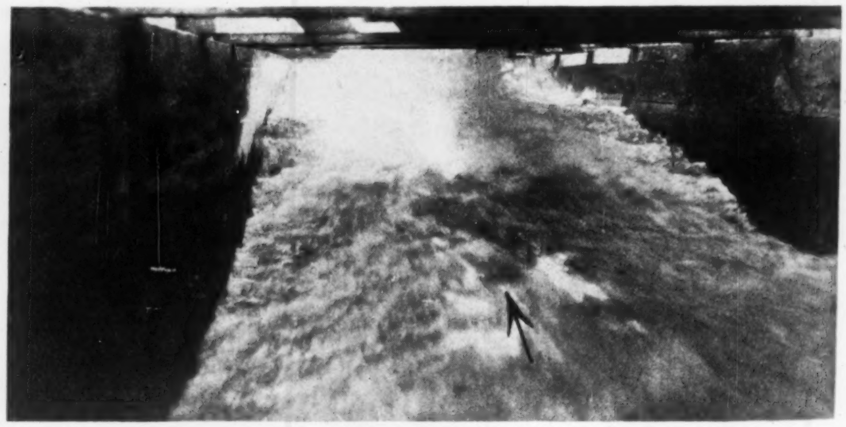
Other unusual flow conditions of interest to the designer and operator are related to the various types of waves set up. These usually have a direct bearing on the amount of freeboard available within a given structure. There are several types of waves found in flowing water. On steep inclines, moderate flows will be characterized by reaches of dark shooting water with over-running waves like small tidal bores. Such waves travel much faster than the main flow. Some move more rapidly than others, and catch up with three or four slower waves in a distance of from 600 to 800 ft. Again, where water is running in a lined canal or flume at a velocity close to critical, a series of stationary waves may be set up by some obstruction and may persist in form and location for days at a time.

Related to these waves are the heavy periodic surges that come down some chutes when they are running to capacity. Such surges, however, do not over-run the main body of water but appear to synchronize with it. Likewise there are several types that persist as stationary waves, even when the velocity is faster than the critical, both above and below the wave. These, of course, are entirely different from the hydraulic jump usually existing at the transition between flows faster and slower than Unwin's critical velocity.

Waves down chute structures are of interest to the engineer for two reasons. They control the freeboard at a mean depth several feet less than would hold if the flow were smooth; and they set up a violent surging in the pool below the chute, and thus cause excessive erosion.

FLOW PHENOMENA THROUGH SHORT CONSTRICTIONS

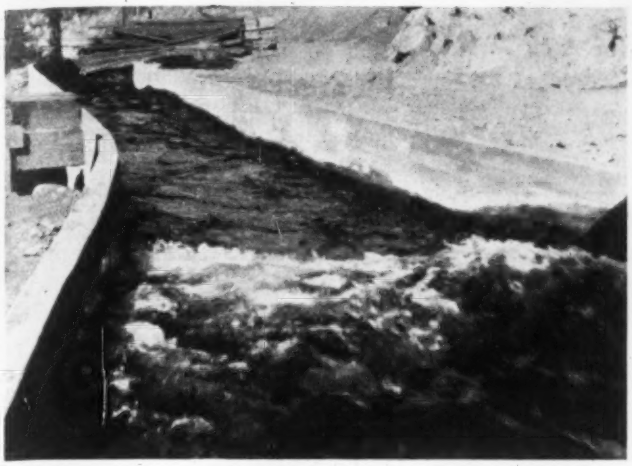
Flows through short constrictions have been greatly neglected in standard works on hydraulics, although these structures are very common in irrigation and hydro-electric systems. They comprise culverts, short tunnels, short flumes, check boxes and, in fact, all structures that constrict the water prism and require an increase in the velocity of the water for a short



Hydraulic Jump in a Power Flume

The water in the foreground has come down a steep chute, generating velocities higher than Unwin's critical. From this point down, the slope of the structure is not sufficient to maintain the "shooting stage," therefore the jump occurs, causing some of the water to surge out of the flume.

distance. Being of limited length, the constructed slope of the structure cannot be used in the solution of a standard flow formula to determine the hydraulic elements. The structure may be set without grade or even inclined upward in the direction of flow without material difficulty. The water-surface line is determined by the energy content available at any point in the structure and by the stage of the water in the outlet pool, except in cases where this stage is below the surface at critical depth in the outlet end of the constriction. When this is the case, the outlet brink becomes a control and the flow in the constriction is independent of the stage in the pool below.



Stationary Waves Easily Mistaken for Hydraulic Jump

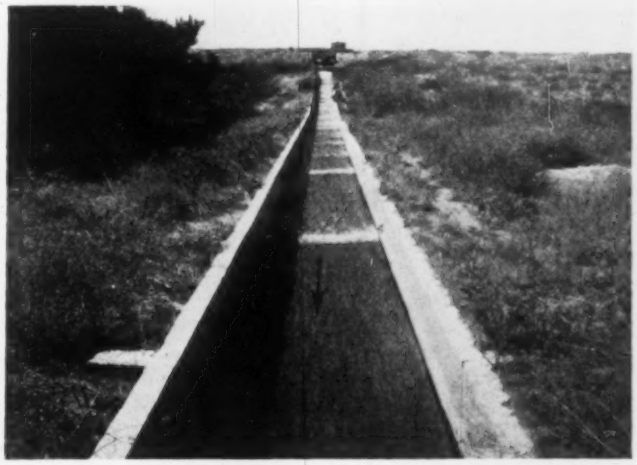
For a true jump, the velocity upstream must be greater than the critical. It is here materially slower than this, and the wave indicates only an abrupt recovery of velocity head at an outlet.

Flow in a Concrete-Lined Chute

Where the canal approaches the chute without curvature, the flow in the chute is quite even. Air is being gradually entrained at the sides of the water prism. The section of the chute in the foreground is steep enough to maintain a flow faster than the critical velocity, but not so fast as that at the foot of the upper section.

Short constrictions, where the water is accelerated in an inlet transition, carried at a high velocity for a short distance, and decelerated in an outlet transition, have been much neglected in the past. Velocity in an open channel can only be increased at the expense of head of elevation. This fact has been overlooked in some structures. A design based on, say, Kutter's formula, would care for the flow after it had been established, but would not provide for generating the high velocity called for by the attendant elements. Likewise, recoveries of velocity head are perfectly feasible as a rule, but this fact has often been neglected. Kinetic energy, unchanged to potential by recovery, wears itself out in the erosion of banks, to the injury of a canal system.

Flow formulas such as Kutter's and Bazin's are based on the assumption that the channel under consideration is of sufficient length to develop a normal or neutral flow. That is, the surface will gradually become approximately parallel to the bottom; the velocity will become uniform throughout the length considered; and the energy gradient, which is the true measure of the various losses, will be parallel to the water surface and the bottom. Obviously, except by coincidence, this uniform condition does not hold within a short constriction and the problem must be attacked in other ways. It might be added that, in such a short constriction, the energy gradient is hardly ever found parallel to the bottom of the structure or to the water surface. Usually the bottom is a straight line, in profile, but the water surface and the energy gradient are both curved in profile but are not parallel. All these elements complicate the problem.



Over-Running Waves, Typical of Steep Inclines

Such waves, catching each other, and all running faster than the main sheet of water, cause heavy erosion by periodic splashes in the outlet pool, a condition most noticeable for the smaller flows. As the volume of water is increased, this action is less marked and may be replaced by heavy periodic surges traveling at the same rate as the main flow.

RECOVERY OF VELOCITY HEADS IN TRANSITION SECTIONS

Flow through short constrictions is greatly influenced by inlet and outlet transition sections, since these occupy a considerable part of the total length of the structure. Experience has shown that water will largely shape its own transition at the inlet end of a structure. The inlet loss, regardless of the type of transition, is but a small percentage of the difference between the velocity heads which obtain for the velocities in the leading channel and in the constriction proper. In other words, water may be accelerated with minor losses.

However, any deceleration in velocity, such as occurs at the outlet end of constrictions, is accompanied by a material loss. We have found that the old policy of neglecting any possible recovery of velocity head should be discontinued. With moderately well shaped outlet transitions, from 50 to 75 per cent of the head lost at the inlet, in creating the velocity required through the structure, may be recovered at the outlet.

Some City Planning Problems of Tacoma

A Progressive Northwestern City Profits by Experience

By C. E. PUTNAM
MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
CITY ENGINEER, CITY HALL, TACOMA, WASH.

LOCATED on the shores of beautiful Puget Sound, with wooded ravines extending back from the water, Tacoma has a natural setting which might well excite the enthusiasm of any city planner. Yet, in common with many other American cities, it has had difficulty in adopting and carrying out a unified plan to stimulate present tendencies to growth and provide for future expansion. In this abstract of his address delivered before the City Planning Division of the Society at the Tacoma Convention, on July 9, Mr. Putnam describes Tacoma's difficulties and successes in city planning.

SURROUNDED on three sides by the waters of Puget Sound, with the snow capped peak of Mt. Tacoma (Mt. Rainier) to the east and the Olympic Mountains to the west, the City of Tacoma has a setting for the building of a city to be found in no other part of the world. Among its recreation centers, Wright Park, McKinley Park, and Lincoln Park are good examples of landscaping. Then there is Point Defiance Park, which, with its acres of natural forest of giant fir, cedar, and hemlock, its cool, pleasant boulevards, and the boating, fishing, and bathing in the waters of Puget Sound, is well worth a journey of many miles.

Very little, however, has been accomplished in the carrying out of a broad scheme of city planning. From time to time a feeble attempt has been made to work out such a plan, but lack of an educational campaign among the citizens and of a spirit of cooperation among the civic organizations of the city has been the great difficulty in the past.

Naturally, the framework upon which the whole scheme of city planning depends is the street system. Tacoma has been fortunate in its streets and boulevards, for in spite of the lack of a comprehensive plan, there is a framework which fits its needs very well. The streets are laid out on a rectangular plan, true north and south and east and west, except for some variation in the business section and in the vicinity of the Stadium and the Seminary districts.

Here the streets lie northeast and southwest, so that the main arteries run with the contours.

Although at first glance the visitor may be disturbed by some of the steep grades, he finds after a study of the street layout that there are easy grades connecting all parts of the city, along streets which run diagonally up the hillsides.

The most noticeable drawback in the business section is that the blocks are of too great length, so that there is an absence of business corners and a slight tendency to traffic congestion caused by the lack of cross streets. In order to remedy this situation, plans have been completed to open up South 10th and 12th streets from Pacific Avenue to Tacoma Avenue, at a cost of about $850,000. It is also proposed to cut through another street about a mile in length running diagonally up the hill from Pacific Avenue and South 24th Street to South 12th Street and Tacoma Avenue. These streets practically converge in the vicinity of the Court House, where there is an excellent location for a civic center overlooking the city and the harbor.

Many of the streets in the older section of the city have 24-ft. paved roadways, built in the days of the horse and buggy, but fortunately there is ample room for widening by encroaching on the parking strips, many of which are 25 ft. and over in width. This work will start in 1932, when the Point Defiance Boulevard will be widened to 36 ft.

ZONING ORDINANCES PASSED

In 1919 the city attempted to put into effect a zoning ordinance which was without doubt one of the saddest examples of the result of a lack of knowledge and foresight in city planning in the history of the city. The ordinance was drawn up by men with no knowledge of planning; no study was made of values, use districts, fire protection, transportation, or other questions that should have been taken into consideration. Still less was accomplished by redrafting this destructive piece of legislation in 1927. The revised ordinance designated the main business section as a wholesale, retail, and manufacturing section, and zoned every street upon which a street car operated, and almost every boulevard, as a retail district. Many of these sections were miles in length.

As a result, real business property as well as residence property suffered considerable loss in value, which could have been avoided. Out of this chaos, as no doubt has been the case in other cities, many

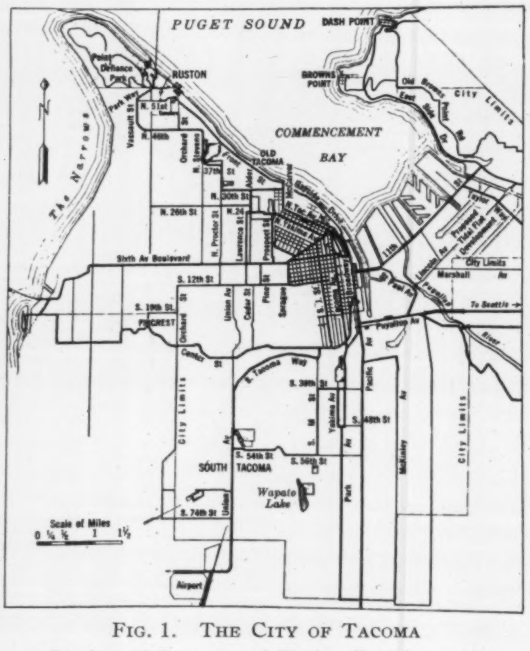

PUGET SOUND
DASH POINT
Point Defiance Park
RUSTON
BROWNS POINT
Old Browns Point Rd
East Side Dr
City Limits
The Narrows
Park Way
Vassault St
N. 51st
N. 46th
Orchard St
N. Stevens
Front St
N. 37th St
OLD TACOMA
COMMENCEMENT BAY
Alder St
N. 30th St
N. 26th St
N. Proctor St
N. 24
Lawrence St
Prospect St
McCarver
Ruston Way
Bayside Drive
N. Tac. Av
N. Yakima Av
Taylor Way
Proposed Tidal Flat Development
11th
Lincoln Av
City Limits
Marshall Av
Sixth Av Boulevard
S. 12th St
Pine St
Sprague
S. 19th St
FIRCREST
Orchard St
Union Av
Cedar St
S. J. St
Broadway
St Paul Av
Puyallup Av
To Seattle →
River
Center St
S. Tacoma Way
S. 38th St
Pacific Av
S. M St
Yakima Av
S. 48th St
City Limits
Av
S. 54th St
SOUTH TACOMA
S. 56th St
Wapato Lake
Union
S. 74th St
McKinley Av
Park Av
City Limits
Scale of Miles
0 ¼ ½ 1 1½
Airport

FIG. 1. THE CITY OF TACOMA
Boulevard System and Harbor Development

efficient and prosperous retail districts have grown up and are operating under ideal conditions, due to the civic pride and foresight of certain community leaders.

It is on the development of these districts that the present city planning commission has had to base the new zoning ordinance, for existing tendencies must be followed. The new ordinance divides the city into six classes of use districts, stating very clearly the particular use of each district; the class of buildings; their uses; height limitations; front, side, and rear yards; lot areas; and setbacks. The use districts in the new ordinance are residential (Classes 1 and 2), local business, commercial, manufacturing, and unrestricted. This ordinance was formulated after a thorough study by the planning commission of the needs of the city, and after consultation with many of the men best informed on such matters.

PROVISIONS FOR DRAINAGE

Combined storm and sanitary sewers provide for the drainage of Tacoma. The system was built up piecemeal, partly by assessment against property and partly by general funds, with very poor results. The revised city charter, drawn up in 1927, ordered a comprehensive survey of the city to devise a general system of sewerage and drainage that would effectively take care of future growth. Engineering studies were carried on over a period of two years, with the result that a very complete system has now been laid out. Although it is financially impossible to carry out the entire plan at once, the system is gradually being built up with some intelligent provision for future needs. All facilities will be of sufficient size to adequately care for present run-off and for normal growth in population during a period of at least 20 years.

The question of disposal has been thoroughly investigated. For many years to come sewage and run-off will be picked up by an intercepting sewer on the waterfront and discharged into the bay at a point where sufficient depth has been found. After a careful investigation of tides and currents, by floats, it has been determined that no nuisance will be experienced through fouling of the shore lands for some time. In the general scheme proposed, provision has been made for the possibility that greater population density and an insufficient distribution of the effluent may make a sewage disposal plant necessary. No expense has been spared in assembling the necessary data and retaining expert consultants to evolve a comprehensive plan.

WATER, LIGHT, AND POWER

No city plan is complete unless it provides for an ample supply of good water, not only for the present but also for the future. This problem has been very effectively solved by the City of Tacoma. Now that the demand has grown to the capacity of the gravity water system, the Water Department has started the development of wells in the southern part of the city. Here a great body of water has been tapped, which, according to data on hand, is several hundred feet deep and will give an adequate supply for many years to come.

In supplying electric power and light and in satisfying the increasing demand for these utilities, Tacoma has gone far. It has obtained not only a great reserve of hydro-electric power but also auxiliary steam power plants of sufficient capacity to carry on in case of a temporary shortage of water. Water, light, and power development are the only phases of the city plan that have been carried out in a very effective way.

PORT DEVELOPMENT STUDIED

In order to obtain expert advice on harbor development, in 1911 the City of Tacoma employed the late Virgil G. Bogue, M. Am. Soc. C.E., and later accepted his very economical and feasible development scheme, containing advice on the location of streets, railroads, waterways, and public utilities. From the day they were accepted. these plans have been carefully preserved in the city archives as evidence of good intentions that were never carried out.

In 1930 some one happened to remember that such a study had been made, but by that time an entirely different development was in progress. Railroads were not where they should be, and existing streets and highways worked at cross-purposes with the waterways. In order to remedy this situation and adhere to the original plan, expensive drawbridges would have been necessary and the result would have been constant interruptions to both land and water traffic.

Finally W. A. Kunigk, M. Am. Soc. C.E., Superintendent of the Water Department, offered a practical modified plan, which was accepted. The first unit of this development, the dredging and bulkheading of Hylebos Waterway, on the extreme east side of the tidal flats, is now in process of construction. This was the only portion of the Bogue Plan that could be used. From this waterway there will be constructed waterways running east and west, instead of north and south as proposed by Mr. Bogue. Since all land traffic and service to the flats is now east and west, such development is more feasible and economical than the original layout.

These facts show just what the city planning problems of Tacoma have been and are. The city has found it a costly mistake to plan for the future and then fail to carry out the plans. This fact is now realized and from now on more care will be taken to work toward a definite plan.

CITY PLANNING COMMISSION APPOINTED

The City Planning Commission of Tacoma consists of the mayor, the city attorney, and the city engineer ex-officio, with seven appointed members. The president of the commission is Major General Alexander. All the members were appointed by the mayor, who selected men of recognized ability, leaders in civic affairs, having a knowledge of city planning.

In order that city planning may take its proper place in the development of Tacoma, the first work of the planning commission and the city officials will be the education of the people to believe that it is not an imposition on individual rights to require adequate sanitation, to limit the heights and areas of buildings, or to determine the uses of districts by zoning. A great part of the work of the engineer and the student of political economy lies in the awakening of city dwellers to their responsibility for the making or marring of their city's future. By city planning alone can stabilization of values and harmony of development be secured.

Four Modern Hydro-Electric Power Developments

Recent Projects Supply Additional Power to Puget Sound Region

BECAUSE of its abundant rainfall and high elevations, the Northwest is well adapted to the generation in large blocks of cheap hydro-electric power. The largest centers of use for this energy are the cities of Portland, Seattle, and Tacoma, which are reaching out into the nearby mountains to secure power and light for their homes and factories. Of outstanding interest are the four hydro-electric power developments which were described at the Tacoma Convention, before the joint meeting of the Power and Construction Divisions on July 9.

As Mr. Gongwer explains, Tacoma has this year completed the second unit of its Lake Cushman Development on the Skokomish River, to generate the final 75,000 hp. available on the North Fork.

Seattle is developing the Skagit River, where, as Mr. Carver states, 1,200 ft. of head and 1,000,000 hp. are available at a load factor of 40 per cent. The Rock Island Plant on the Columbia River is described by Mr. Shannon. This low-head development, to consist of twelve 21,000-hp. units, is in process of construction. The construction of the Ariel Dam on the Lewis River, described by Mr. Griswold, is the first step in the complete utilization of that stream by Portland.

Tacoma Completes Second Cushman Plant

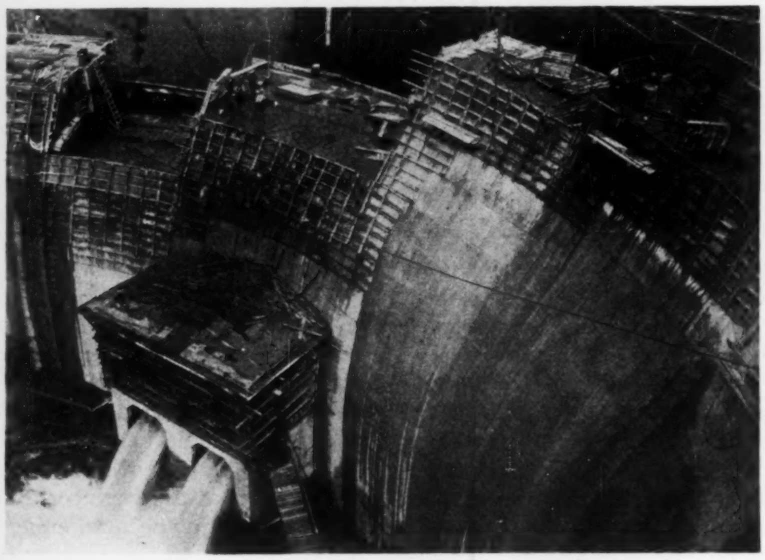
By VERNE GONGWER

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS

SUPERINTENDENT OF HYDRAULIC DESIGN AND CONSTRUCTION, DEPARTMENT OF PUBLIC UTILITIES, CITY OF TACOMA

WITH its watershed lying on the east slope of the mountains of the Olympic Peninsula and its upper reaches fed by almost perpetual snows, the Skokomish River is a tributary of the Puget Sound basin. The average annual run-off from the 91 sq. miles of its watershed for the years of record is approximately 100 in., representing an average flow of 775 sec-ft., or approximately $8^1/_2$ sec-ft. per sq. mile. The location of the river as related to the City of Tacoma can be seen by examining Fig. 1 on page 1107.

The first step in the Cushman Development consisted of the construction of Cushman Dam No. 1, an arch dam 280 ft. high, which created a storage reservoir having a capacity of 440,000 acre-ft., sufficient to fully regulate the flow of the stream. Connected with the dam by a short concrete-lined tunnel through rock, is Power House No. 1, with 50,000 hp. of installed capacity in two identical units, operating under a gross head of 255 ft. at full reservoir. The spillway level is 735 ft. above mean sea level, the water being returned to the stream at a tailrace elevation of 480 ft.



CUSHMAN DAM NO. 2 DURING CONSTRUCTION
Butterfly Valves Discharging

NORTH FORK OF SKOKOMISH RIVER COMPLETELY DEVELOPED

Power House No. 1 was placed in service in 1926. After an interval of about one year, during which a material increase in load indicated the approaching need for further installation, the preparation of detailed plans for Plant No. 2, ultimately to contain three 37,500-hp. units, were begun. Contracts were let in the spring of 1929 and the first unit was placed in service on January 1, 1931. This event marked the full development of the North Fork of the Skokomish River by the City of Tacoma.

As shown in the general plan and profile, Fig. 1, Cushman Power Plant No. 2 consists of Cushman Dam No. 2, constructed $1^3/_4$ miles below Power House No. 1, to impound water again at tailwater level of the first plant, and divert it through Cushman Tunnel No. 2 to a point on the bluff above Hood Canal, where it drops through steel penstocks to Power House No. 2, to develop the remaining average gross head of 480 ft. to tidewater.

Reservoir No. 2 has a total capacity of 7,300 acre-ft., the intended range of the drawdown being 20 ft. Thus 2,000 acre-ft. of pondage are available, an amount equal to nearly one day's water demand for the three 37,500-hp. units in Power House No. 2, at the usual load factor.

The canyon at the dam site is comparatively wide at the bottom, with walls composed of rock which was known to be seamy and jointed to a considerable extent. For these conditions a simple arch-type dam of one center, with an upstream radius of 140 ft., was selected. A comprehensive exploration with diamond-drill test borings was made and revealed that bedrock lay approximately 70 ft. below the stream bed and that the abutments were somewhat jointed and fissured.

From the lowest foundation point to the top of the parapet, the maximum section is 240 ft. high. It has a thickness of 8 ft. under the roadway and a nominal thickness of 40 ft. at the base. An additional downstream flare below the stream bed renders the base actually 50 ft. thick. The dam contains 38,000 cu. yd. of concrete.

Siphon-Controlled Drum Gates in Operation, Dam No. 2
Three Gates Regulate Reservoir Level to Within One-Tenth of a Foot of Desired Elevation

Thrust blocks designed for the ends of the arch, when finally excavated to satisfactory rock, became each 50 ft. in maximum height. The eastern thrust block, being exposed to water pressure, was designed as a gravity section. The additional concrete required to take the water thrust was added on the reservoir side because of the topography, and in order to preserve the symmetrical appearance of the downstream elevation of the structure.

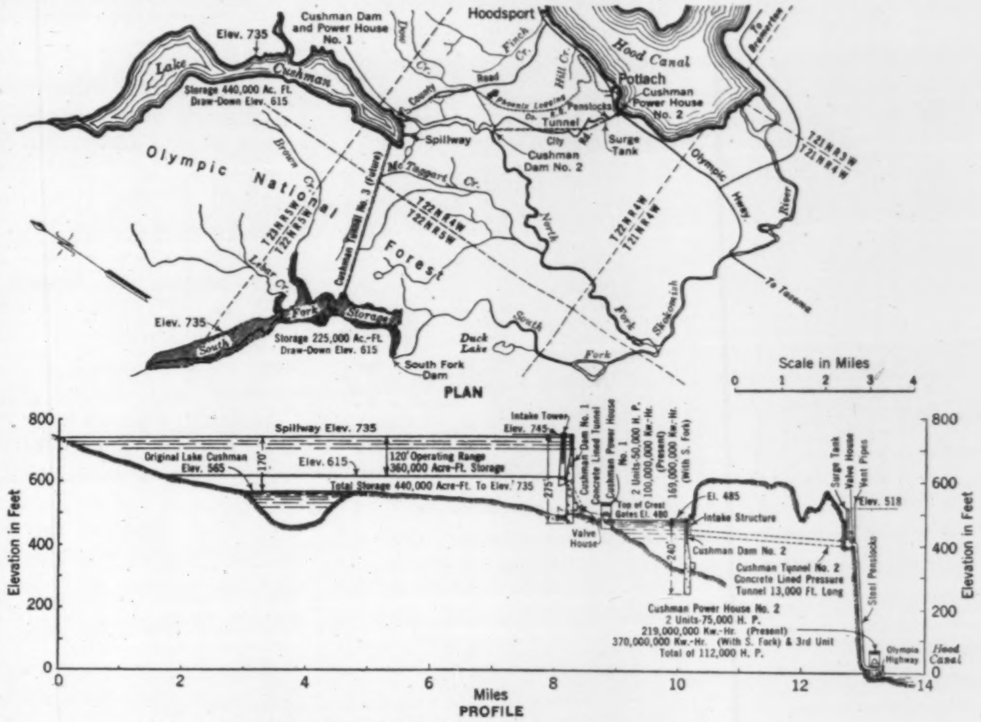

Cushman Dam and Power House No. 1
Elev. 735
Hoodsport
Lake Cushman
Storage 440,000 Ac. Ft. Draw-Down Elev. 615
Dow Cr.
Finch Cr.
Hood Canal
To Bremerton
County Road
Hill Cr.
Potlach
Cushman Power House No. 2
Phoenix Logging Co. R.R.
Penstocks
Tunnel
City
Surge Tank
Cushman Dam No. 2
Spillway
Olympic National Forest
Brown Cr.
McTaggart Cr.
Cushman Tunnel No. 3 (Future)
T23NR5W
T22NR5W
T22NR4W
T22NR5W
T22NR4W
T21NR4W
T21NR3W
T21NR4W
Olympic Hwy.
River
North Fork Skokomish
To Tacoma
Lebar Cr.
Elev. 735
South Fork Storage
Storage 225,000 Ac.-Ft. Draw-Down Elev. 615
South Fork Dam
Duck Lake
South Fork
Scale in Miles
0 1 2 3 4
PLAN
Spillway Elev. 735
Intake Tower Elev. 745
Original Lake Cushman Elev. 565
170'
Elev. 615
120' Operating Range 360,000 Acre-Ft. Storage
Total Storage 440,000 Acre-Ft. To Elev. 735
275'
Cushman Dam No. 1
Concrete Lined Tunnel
Cushman Power House No. 1
2 Units-50,000 H. P.
100,000,000 Kw.-Hr. (Present)
169,000,000 Kw.-Hr. (With S. Fork)
Top of Crest Gates El. 480
Valve House
El. 485
Intake Structure
240'
Cushman Dam No. 2
Cushman Tunnel No. 2 Concrete Lined Pressure Tunnel 13,000 Ft. Long
Surge Tank
Valve House
Vent Pipes
Elev. 518
Steel Penstocks
Cushman Power House No. 2
2 Units-75,000 H. P.
219,000,000 Kw.-Hr. (Present)
370,000,000 Kw.-Hr. (With S. Fork) & 3rd Unit
Total of 112,000 H. P.
Olympic Highway
Hood Canal
Elevation in Feet
800
600
400
200
0
0 2 4 6 8 10 12 14
Miles
PROFILE

FIG. 1. Lake Cushman Development, Skokomish River

A roadway $12^1/_2$ ft. wide occupies the deck of the dam, with provision at the spillway end for a bridge, should it ever become expedient to construct a road through to the South Fork Project. A concrete-lined spillway with a capacity of 30,000 sec-ft. is provided through a rock cut around the western end of the dam.

The partially depleted stage of Reservoir No. 1 protected the foundation excavation from floods during construction. Diversion was made by means of a timber crib dam with a steel sheet-piling cut-off 30 ft. deep. The discharge from Power House No. 1, which reached a maximum of 2,000 sec-ft., was passed through a timber flume, 10 by 20 ft., supported by timber trusses spanning the excavation. The flume and the diversion dam were constructed during a period when it was feasible to close down Power House No. 1 for maintenance work.

A large number of grout holes were drilled with diamond core drills, including holes under the spillway weir and across the tunnel intake. After the placing of from 10 to 20 ft. of concrete, the holes were grouted at a pressure of approximately 150 lb. per sq. in. Many of these holes were deep, but few took more than a nominal amount of grout. Very little intercommunication of holes was observable.

The usual type of keyed vertical contraction joints, with beaded copper water stops upstream and downstream, were spaced at approximately 30-ft. intervals. Horizontal copper water stops were also used at the upstream face to seal the joints between the daily pours.

Concrete aggregates were brought in by barge and hauled 4 miles to the site. A nominal mix of 1:1.92:3.92 was designed and used with well graded aggregates. This mix, containing approximately 1.25 bbl. of cement per cu. yd., was readily chuted and puddled without appreciable segregation. The resulting concrete is dense and impervious. Test cylinders withstood on the average a pressure of 3,500 lb. per sq. in. after 28 days, and 4,500 lb. after 60 days. A relatively small percentage of reinforcement was placed in both faces of the dam, varied according to stress. It is interesting to note that, in the conception of the penstock piers, concrete was chuted a maximum of 1,400 ft. in a wooden trough. Samples taken at the lower end of the chute tested slightly stronger than those from the same batches taken at the mixer.

CUSHMAN POWER HOUSE NO. 2, AT TIDEWATER ELEVATION
Surge Tank, Vent Pipes, and Penstocks for Two Units

Two outlets 8 ft. in diameter were placed through the base of the dam for unwatering and emergency purposes, each outlet being controlled by two free-discharge, power-operated butterfly valves in tandem.

Individual inclined pipes, each serving approximately 30 sq. ft. of joint area, provide for the grouting of the vertical joints between the sections of the dam. This arrangement permits the pressure and result at each point to be definitely known. The lower 25 ft. of the dam was constructed as a plug, without vertical joints. All contraction joints were carried to foundation rock or to the top of this plug, 50 ft. below the stream bed. Concrete was placed in 5-ft. lifts, the consistency being such that the puddlers, in rubber boots, would usually sink well above their knees. The flume opening was closed by bringing up the section with rapid pours, in forms of especially heavy construction, during a short shut-down of Power House No. 1. Thus any special gate or plug construction was avoided.

The tops of all pours were well broomed until the aggregate stood out, and after the concrete was fully set the surface was chipped where necessary. Immediately before the next pour it was scrubbed and cleaned with water and air and thoroughly inspected.

CUSHMAN POWER PLANT UNIT NO. 1

Height of Dam No. 1—275 ft. (235 ft. above stream bed)
Yards of concrete—90,000
Thickness at bottom—54 ft.
Thickness at top—8 ft., length 470 ft.
Storage capacity—440,000 acre-ft.
Power tunnel—17 ft. in diameter
Penstock tunnels—12 ft. in diameter
Head—255 ft. to 135 ft.
Average river flow, North Fork—800 sec-ft.
Power house concrete structure—74 by 134 ft.
Turbine capacity—50,000 hp.
Speed—200 r.p.m.
Length of transmission line—44 miles
Voltage—110,000
Lines cross the "Narrows" of Puget Sound by longest span in the world—6,244.5 ft.
Height of "Narrows" towers—315 ft.
Weight of each tower—150 tons
Sag of cables—400 ft.
Towers stand on cliffs 325 ft. above sea level.

AUTOMATIC GATES CONTROL POND LEVEL

In order to hold the pond level as high but not higher than the normal tailwater of Power House No. 1, and yet provide ample escape for floods while maintaining and operating under that condition, three 40 by $14^1/_2$-ft. drum-type crest gates are used, patterned after the standard of the U.S. Bureau of Reclamation. The gates are hinged along the upstream horizontal axis and the lower portion of each gate floats in a sealed, water-filled chamber, the height of the gate crest being regulated by the amount of water in the chamber, which is in turn controlled by the siphons. The siphons are fitted with adjustable sniff pipes for breaking the siphon action at any desired stage of reservoir level.

Although siphons have been used for the operation of drum gates in a number of installations, the improved design of the siphons employed in this development and the operating results obtained are believed to be original. By a close setting of the range between the priming and the breaking point of the siphon action, the gates become perfectly steady and do not oscillate on their hinges. At the same time they remain immediately responsive to the slightest change in reservoir level, and regulate the pond to a range as small as one inch. Wear on the hinges and seals is thus reduced to a minimum.

The ends of the 3-in. sniff pipes are then never submerged, but the water level is held slightly below them and they constantly draw in a mixture of air and water, the moisture content of which varies as the reservoir tends to rise or fall away from the ends of the pipes. The siphon action, which under these conditions is continuous, is assisted or impaired as the mixture brought in by the sniff pipes varies in moisture content. Consequently water is discharged from beneath the drum gates at a greater or less rate than it is supplied by the inlet gates in the particular throttled position at which they are set. Within a moderate range the setting of the inlet gates is not critical, since a further tendency to balanced action is present in the differential head in the inlet and siphon wells respectively.

PRESSURE TUNNEL REINFORCED AND LINED

To transmit the water from Dam No. 2 by the most direct and economical route to an advantageous point above tidewater, it was necessary to tunnel beneath a table-land of typical glacial drift, consisting of sand, gravel, and hardpan, in miscellaneous stratification and arrangement.

A finished inside diameter of 17 ft. was chosen for the tunnel, on the basis of lowest annual cost and power loss.

This decision was made after a study of curves plotted from the results of a large number of estimates for various diameters. The probability of a diversion of the South Fork of the Skokomish into Reservoir No. 1 was also taken into consideration.

In this study, various sizes and types of surge tanks and forebays, with cost and corresponding amplitude of positive and negative surges, were considered. On these factors depended the amount and cost of the steel reinforcement, which alone came to $250,000.

The water demand for the ultimate installation of the three 37,500-hp. units in Power House No. 2 is 2,700 sec-ft. This would produce a velocity at full load of 12.5 ft. per sec., with an estimated total friction loss of between 20 and 22 ft.

The materials through which the tunnel was to be cut were considered to be fairly firm, but there was no logical means of forecasting where unequal pressures might occur. The tunnel was therefore lined with a minimum of 8 in. of concrete inside of timbers and reinforced for full bursting pressure at maximum surge. Square deformed reinforcing bars were used, bent to shape in the tunnel, and spaced from 5 in. on centers at the lower end of the tunnel to 12 in. on centers near the intake. The longitudinal steel is continuous through the construction joints.

Under static conditions the center line of the tunnel is $36^{1}/_{2}$ ft. below the maximum reservoir level at the intake and 69 ft. below it at the surge tank. At this tank the maximum head is 104 ft. at maximum surge. Allowing 20 ft. of operating range in the reservoir, the top of the tunnel has a minimum submergence of 8 ft.

On account of the severe specification requirement that all back-fill between the temporary and permanent timber and the over-break be compacted and grouted, the contractor attempted the use of steel poling boards. These were pried along by hand over removable steel arch rings, the timber lagging being installed tightly beneath the poling boards, with practically no over-break.

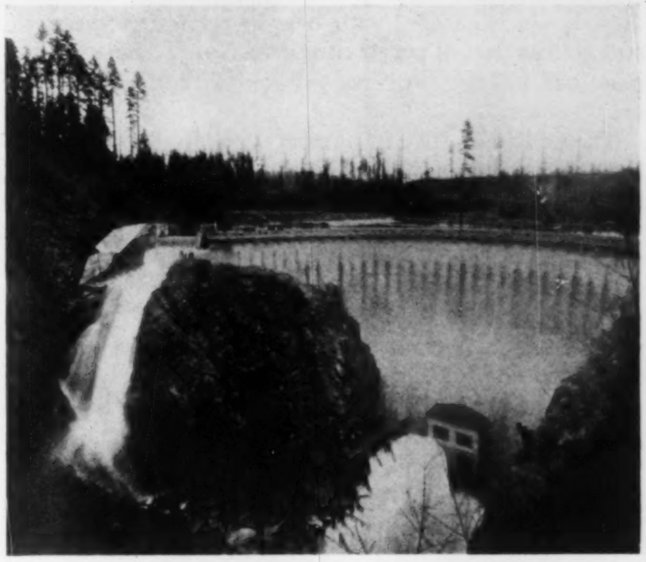
This process had great possibilities and about 1,500 ft. were driven in this way. However, after several sections of loose ground had been passed, the method was abandoned, principally because of apprehension among the workmen, to whom it was unfamiliar. It was believed that the cost of backfilling and the greater expense of grouting would be offset by more speed with the top-center drift. However, the average speed with both methods was about the same.

Cushman Power Plant Unit No. 2

Height of Dam No. 2—240 ft. (160 ft. above stream bed)
Yards of concrete—38,000
Thickness at bottom—40 ft.
Thickness at top—8 ft.
Length—330 ft.
Diversion tunnel diameter—17 ft., length 13,000 ft.
Penstocks, diameter—10 ft. 6 in. to 9 ft. 0 in.
Differential surge tank—Height 94 ft., diameter 65 ft.
Power house concrete structure—81 *by* 180 ft.
Turbine capacity—75,000 hp. (provisions made for 37,500 hp. more)
Head—480 ft. to 460 ft.
Speed—300 r.p.m.
This unit uses the same water that is discharged from Unit No. 1.

The concrete lining contains 1.33 bbl. of cement per cu. yd. and was placed by pneumatic equipment in complete 40-ft. sections in one operation, without longitudinal joints. For this purpose, special forms and supporting girders were invented by the contractor.

After completion of the lining, the gravel backfilling between the temporary and permanent timber, and all voids and lost ground were filled practically to refusal with 1:3 cement grout. In the arch, six grout pipes were provided for each 40-ft. section. Additional pipes were installed at the spring lines and elsewhere as conditions indicated. Concrete guns having a capacity of 2 cu. yd. and an air pressure of approximately 110 lb. per sq. in. at the gun, were employed for this purpose. While the original undisturbed ground would take practically no grout, a large amount was required at points where ground had been lost.



Cushman Dam No. 2 Completed
Drum Gates and Butterfly Valves Discharging

Cut-off collars around the lining were constructed at approximately 1,500-ft. intervals by driving $1^{1}/_{2}$-in. pipes back into the ground several feet and grouting to refusal with 1 : 2 grout, followed by neat cement. Copper water stops were specified in all longitudinal and circumferential construction joints.

These precautions have apparently been effective in promoting water-tightness and in preventing seepage along the lining. Underdrains installed at the adit and lower portal show but minor seepage, an amount approximately equal to the groundwater which was encountered near these entrances. Probably this is the first tunnel where copper water stops have been employed. Results are highly satisfactory and appear to show a positive solution for preventing joint leakage.

Comparative estimates and considerations of safety resulted in the selection of a steel surge tank. The differential type was chosen because it gives rapid regulation and is smaller in size. So far as is known, this is the largest differential surge tank yet constructed. It is 65 ft. in diameter and 94 ft. high, with a riser $14^{1}/_{2}$ ft. in diameter. The bottom course of plates is $^{15}/_{16}$ in. thick. The concrete foundation is integral with the encasement of a steel liner, 17 ft. in diameter, which passes entirely under the surge tank and extends back into the tunnel approximately 100 ft. A steel manifold with three branches 10 ft. 6 in. in diameter, one blanked for future expansion, is attached immediately downstream from the surge tank foundation. It is encased with heavily reinforced concrete, and acts as part of penstock anchor No. 1. Between the manifold and the penstocks, 10-ft. 6-in. butterfly valves are installed.

The penstocks, two in number, vary by 3-in. intervals from a diameter of 10 ft. 6 in. at the top, to that of 9 ft. 0 in. near the base of the hill. The plate varies from $^{7}/_{16}$ in. to $1^{1}/_{8}$ in. in thickness. These penstocks were designed at ordinary working stresses for a 50 per cent excess over the static pressure, measured to the top of the surge tank, although 100 per cent relief valves are provided at the turbines. The slamming of the butterfly inlet valves at the turbines would produce a momentary pressure approximately equal to the elastic limit of the pipe. Between the anchors sleeve-type expansion joints were installed, with square flax packing and alemite lubrication. These joints have practically no leakage. The pipes were tested in sections by pumping to 35 per cent over working pressures. The surge tank was pumped full.

Power House No. 2 is situated near the foot of the hill, 130 ft. back from the center line of the Olympic Highway, on the bank of the Hood Canal. The fact that the discharge of the South Fork of the Skokomish is somewhat more than half that of the North Fork and is likely in the future to be brought into Reservoir No. 1 by a four-mile tunnel, favored the adoption of three identical generating units for the ultimate development, two to be installed at this time.

The 27,000-kw. (37,500-hp.) units are of the vertical direct-connected type, with turbines $7^{1}/_{2}$ ft. below the main floor level. The generators are mounted on structural steel supporting barrels resting on the cast-steel scroll cases. The concrete pedestals which were cast around the supporting barrels have flaring tops which form walkways at mezzanine level.

As a precaution against the possibly injurious action of salt water, the center line of the distributors is 2 ft. above extreme high tide. The length of draft is limited at low tide by a weir 70 ft. long across the tailrace, at an elevation 14.5 ft., below the center of the distributors. Of course the actual draft at low tide varies with the load and is less than this amount by the depth of water passing over the weir. The extreme range of the tide is 17.5 ft. and the weir is 5.0 above extreme low water. However, it was found that the amount of power below the top of the weir was available for so short a time that it justified little or no expense in making the top of the weir adjustable. Synchronous by-pass valves of 100 per cent capacity, connected to scroll cases, are provided.

Foundation excavation extended $25^{1}/_{2}$ ft. below high tide and was performed behind the protection of a low dike, which was constructed to serve also as a detour for the Olympic Highway. One 10-in. pump was kept working at about half its capacity to handle groundwater. No noticeable amount of salt water percolated into the excavation. The power house rests on a 4-ft. reinforced concrete mat bearing upon firm gravel and sand. A steel sheet-piling cut-off was driven along the front edge. The superstructure is of structural steel encased in concrete of classic design.

A 125-ton bridge crane, traveling the length of the main building, was installed and was used for erecting the power house machinery. The heaviest pieces to be handled were the rotors, which weigh 123 tons each and were received completely assembled, on special cars. These were switched from the logging railroad into the end of the power house within reach of the crane.

The outdoor switching and transformer yard is in the rear of the building. Here the current is stepped up from 13,200 volts to 110,000 volts, for transmission to Tacoma. Only one span of conductors was required to make the connection with the original transmission line from Power House No, 1, which was constructed to ultimate capacity at the time of original installation.

The satisfactory completion of the project is a testimonial to the able administrative guidance of Ira S. Davisson, Commissioner of Public Utilities; and is also a most fitting memorial to our late Chief Engineer, Jay L. Stannard, M. Am. Soc. C.E., whose engineering genius conceived the project and carried it nearly to completion before his death in March of this year.

Skagit River Developed by Seattle

By T. H. CARVER

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS

ASSISTANT CITY ENGINEER IN CHARGE OF WATER SUPPLY AND POWER DEVELOPMENT, SEATTLE

SOME twenty miles north of the boundary between the United States and Canada are the headwaters of the Skagit, the largest river flowing into Puget Sound. Below the boundary, the river flows generally southwest for a distance of 125 miles to the Sound. The upper portion, which drains about 1,200 sq. miles, has a rapid fall, and is now being developed by the City of Seattle. At the city's gorge Power Plant the average precipitation is about 74 in. The maximum observed flood in the past 20 years was 55,000 sec-ft. The minimum flow, in severe winter weather is about 450 sec-ft. and the mean regulated flow is about 4,100 sec-ft. On page 1077, Fig. 1 shows the relative location of the Skagit River and the Puget Sound cities.

The development will consist of three major plants: the Gorge Plant, at the lower end of Skagit Canyon, with a proposed installation of about 320,000 hp., a gross head of 375 ft., and nominal storage; the Diablo

Plant, 7 miles upstream from the Gorge Plant, at Diablo Canyon, with a proposed installation of about 320,000 hp. and a gross head of 330 ft.; and the Ruby Plant, 7 miles upstream from the Diablo Plant, below the mouth of Ruby Creek, having a proposed installation of about 480,000 hp. and a gross head of 500 ft. About 60,000 acre-ft. of storage are available to both the Diablo and Gorge plants, and nearly 3,000,000 acre-ft. to all three plants. The total projected installation will provide a little over 1,000,000 hp. and the total available head is 1,200 ft. The development is based on a load factor of 40 per cent.

The first unit of the Gorge Plant has been completed and is in operation. It has three vertical-type generators, with a total rated capacity of 93,000 kw., direct-connected to Francis-type turbines. Water is brought to the wheels through a tunnel 11,000 ft. long, at a head of 270 ft., which will be increased to 375 ft. by the construction of a new dam at the tunnel intake. The generating machinery in this plant will be duplicated in the final installation. Current is transmitted to Seattle, at 165,000 volts, over a single-circuit copper-conductor transmission line carried on wooden poles. The total distance from the Gorge Power House to the North Substation in Seattle is over 100 miles.

Diablo Dam has been completed and is in service to regulate the flow to the Gorge Plant. By this means the capacity of the plant has been increased so that it can meet all power demands until the Diablo Power Plant is finished. The Diablo Dam is 386 ft. high from the lowest foundation to the roadway on top, and about 1,200 ft. long on the crest. It is of the constant-angle arch type and was constructed by the Winston Brothers Company, under contract. Work was started in October of 1927 and finished in November of 1930. There are 325,000 cu. yd. of concrete in the structure, the total cost of which was about $4,000,000.

The first unit of the Diablo Power Plant is now under construction. It will be located about a half mile below the dam, and will be served by a concrete-lined tunnel 2,000 ft. long and 320 ft. square in section. Two steel penstocks will lead from the end of the tunnel to the two vertical-type Francis turbines which are now being installed. The turbines have each a rated capacity of 83,000 hp. at a 330-ft. head, and are direct connected to generators of 63,000-kw. capacity. Current will be generated at 13,800 volts and transformed to 220,000 volts for transmission to Seattle over a double-circuit aluminum-conductor line 125 miles long, carried on steel towers.

Diablo Dam on the Skagit River
Seattle's Municipal Hydro-Electric Development

No actual construction has yet been done on the Ruby Development, but the building of the Ruby Dam, to be over 500 ft. high, will be the next step. The reservoir back of this dam will be 30 miles long, extending 8 miles beyond the Canadian border, and will completely regulate the flow of the river and stabilize the production of power in all the plants below.

Rock Island Development—Columbia River

By William D. Shannon
Member American Society of Civil Engineers
General Superintendent, Stone and Webster Engineering Corporation, Seattle

It was in 1927, when the Puget Sound Power and Light Company was considering the enlargement of its central station capacity, that the Rock Island Hydro-Electric Development had its inception. At that time a detailed study was made of the load curve on the company's system, and the amount of prime power that would be required at various stages in the company's development was estimated.

It was very soon realized that a steam plant was needed most, in order to relieve transmission lines and the then existing hydraulic plants. A steam plant was all the more important for, as it developed, the Northwest was entering a period of climatic change in which lack of rainfall seriously depleted the streams of western Washington. The first unit of the Shuffleton Steam Station was completed on August 28, 1929. At that time the load was growing to such an extent that it seemed advisable to begin construction of the second unit at this station immediately. The present plant, consisting of two 35,000-kw. steam turbines, was completed in the spring of 1930.

In the meantime, studies of various hydro-electric power sites were in progress. Among the sites investigated was that known as Rock Island on the Columbia River. In the fall of 1928, surveys of this site were authorized and they continued through the winter and the following spring. During the winter of 1928–1929, soundings were taken through the ice at Rock Island Rapids, so that a rough topography could be made of the bottom of the river at what later became the dam site. The first topography was used to obtain data for the map exhibits that were filed on December 13, 1928, in an application to the Federal Power Commission for a preliminary permit.

In connection with the final surveys, careful ties were made to all boundary lines and physical structures—including bridges, railroads, highways, buildings, and irrigation pumps—that came within the project boundary or were adjacent to it. Diamond drilling was also carried on at the tentative location of the dam and power house, and further studies were made on all matters which would in any way affect the development. The license for the Rock Island Development was granted by the Federal Power Commission on January 21, 1930.

PHYSICAL CONDITIONS UNIQUE

The Rock Island power site is located some 13 miles below the city of Wenatchee on the Columbia River. It is on the state highway between Wenatchee and Spokane and also on the main line of the Great Northern Railway. Both highway and railroad pass close to the development.

At Rock Island the Columbia River is divided into two channels, known as the east and west channels. Owing to the existence of the volcanic upthrust forming this island, the river broadens out until it has a total width at this point of 2,700 ft. from east to west banks. The power house site is entirely on a huge layer of lava rock which extends all the way across the river and which is stated on eminent authority to be perhaps 1,000 ft. in depth. The river has eroded its way down through this lava, cutting its channels into the softer portions of the basaltic rock. The rock itself is a crystalline formation, very hard, fine grained, and extremely brittle. Some of it takes a polish, making it somewhat resemble what is known as desert varnish. The hexagonal crystals forming the rock vary from 12 to 36 in. across, the average size being in the neighborhood of 20 in.

At Rock Island, the fall of the river where it flows between Douglas and Chelan counties, is somewhat greater than that in most parts of its bed. Its slope averages about 2 ft. per mile, but through the Rock Island district it is about 7 ft. at high water in a total distance of three-fourths of a mile. At low water, the total drop is about 15 ft. Above the Rock Island site the Columbia River has a drainage area of approximately 90,000 sq. miles. The flood season usually begins about April 15 and ends September 15, the flow during this period varying from about 100,000 sec-ft. to a maximum of 740,000 sec-ft., with an average flow of about 350,000 sec-ft. The minimum flow in winter has never been known to go below 21,000 sec-ft.

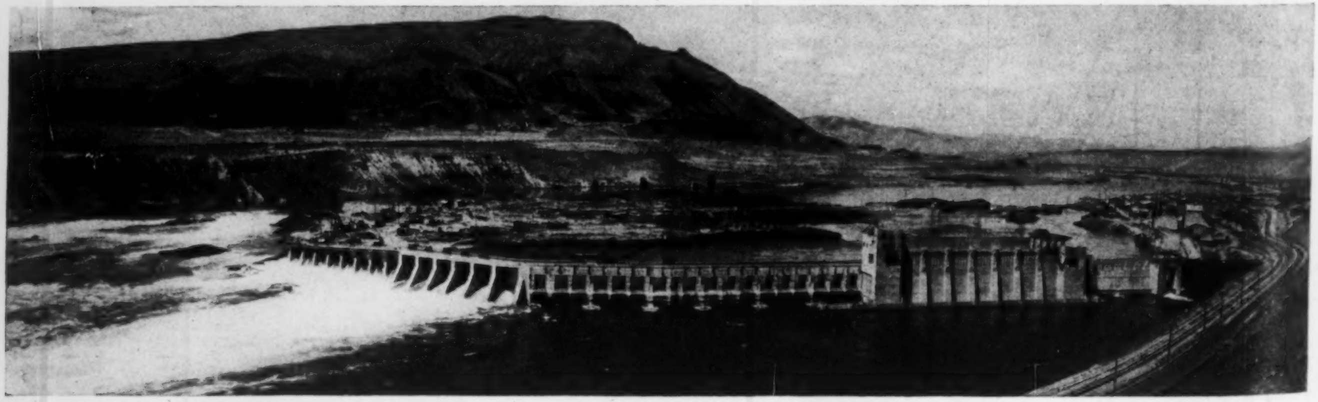
At Rock Island certain physical conditions, such as highways, railroads, and bridges, limited the amount of head that could be developed to a maximum of 51 ft. A dam built to a gate-crest elevation of 600 ft. U.S.G.S., will back up the waters of the Columbia River a total of some 20 miles, or about 7 miles above the city of Wenatchee. Sufficient spillway capacity will be left through the dam so that floods on maximum size will have practically no interference.

Climatic conditions in the Columbia Basin are similar to what one would expect on a high, arid plateau in the North Temperate Zone. The summers are hot and dry, with temperatures reaching 115 deg. fahr., and the winters are dry and at times extremely cold, the temperature going as low as −15 deg. fahr. It is thus seen that in a construction project as large as that at Rock Island it was essential to consider very carefully climatic as well as hydrographic conditions. The placing of concrete in the winter months would ordinarily involve severe handicaps. On the other hand, during the summer, when conditions for construction work are more favorable, the river is in flood and the amount of construction work that can be done is limited.

CONSTRUCTION BEGUN JANUARY 1930

The entire development consists of the following structures: the north abutment, the power house, the east spillway, the west spillway, and the south abutment. They occupy the whole river channel and extend across the island for a total length of 3,500 ft.

After the work of construction was authorized, in January 1930, and after the usual preliminary construction, including housing and plant facilities, it was decided that the first structures to be built should be the cofferdams in the east channel. At that time of year the river was at its lowest stage. By April 15 two large cofferdams, one above the power house site and one below, were ready to withstand the summer floods.



PANORAMA OF THE ROCK ISLAND DAM AND POWER HOUSE
West Channel Cofferdams on Extreme Left, Power House in East Channel, Fishway at Extreme Right

After they were completed and flooded out by the highwater of April, attention was turned to the construction of the north abutment. This part of the dam is on the north side of the river and extends somewhat parallel to it, between the north end of the power house and the state highway. The reason for extending this abutment parallel to the river, in the manner described, was to take advantage of the higher rock elevation and to accommodate the fish ladders, among the largest of their kind ever constructed.

One very interesting phase of the Rock Island Development is the arrangement of these ladders, which were required by the Federal Power Commission at the request of the U.S. Bureau of Fisheries. Two of them are to be built immediately, one on each bank of the stream. They are to be made of concrete or cut out of rock. Each will be about 500 ft. long and 20 ft. wide, laid out on a slope of 1 on 10. The pools will be 10 ft. long, and the ladders will be supplied with water through suitable gates in the abutments.

By August 15 the river had receded to a point where it was near the top of the cofferdams. A 10-ft. lift was quickly put on top of the upstream cofferdam, the structure sealed, and the river blocked off. A similar 10-ft. lift was constructed on the top of the downstream cofferdam, after which the intermediate area was pumped out and the work of excavating for the power house structure was begun. Actual pumping started on August 25, 1930, and was completed five days later. Excavation began on August 30, 1930, and was finished for five of the units on October 13, 1930, so that the work of pouring the foundation slab for the power house could proceed.

The first concrete in the power house structure was poured on September 29, 1930. This work was carried on continuously until March 4, 1931, when both cofferdams were removed and the river was allowed to find its way back through the east channel. During this period of approximately five months, about 150,000 yd. of concrete were poured, and the power house structure for the initial units was carried from the foundations to the roof girders. By March 1931, the generator room was entirely housed in. The power house was then completed by the addition of the two upper stories.

FOUR POWER UNITS TO BE INSTALLED NOW

The power house is a part of the dam and extends about halfway across the east channel. Ultimately it will be 880 ft. in length and extend from bedrock, at elevation 509, to the top of the superstructure, at elevation 671. Foundations have now been finished for all 12 units, but the initial building, which has just been completed, will house only four units of 21,000 hp. each. An addition to the power house can be constructed and new units installed by dropping gates, closing off the river, and pumping out the draft tube. The lower part of this tube and a part of the intake structure have also been built for the eight future units.

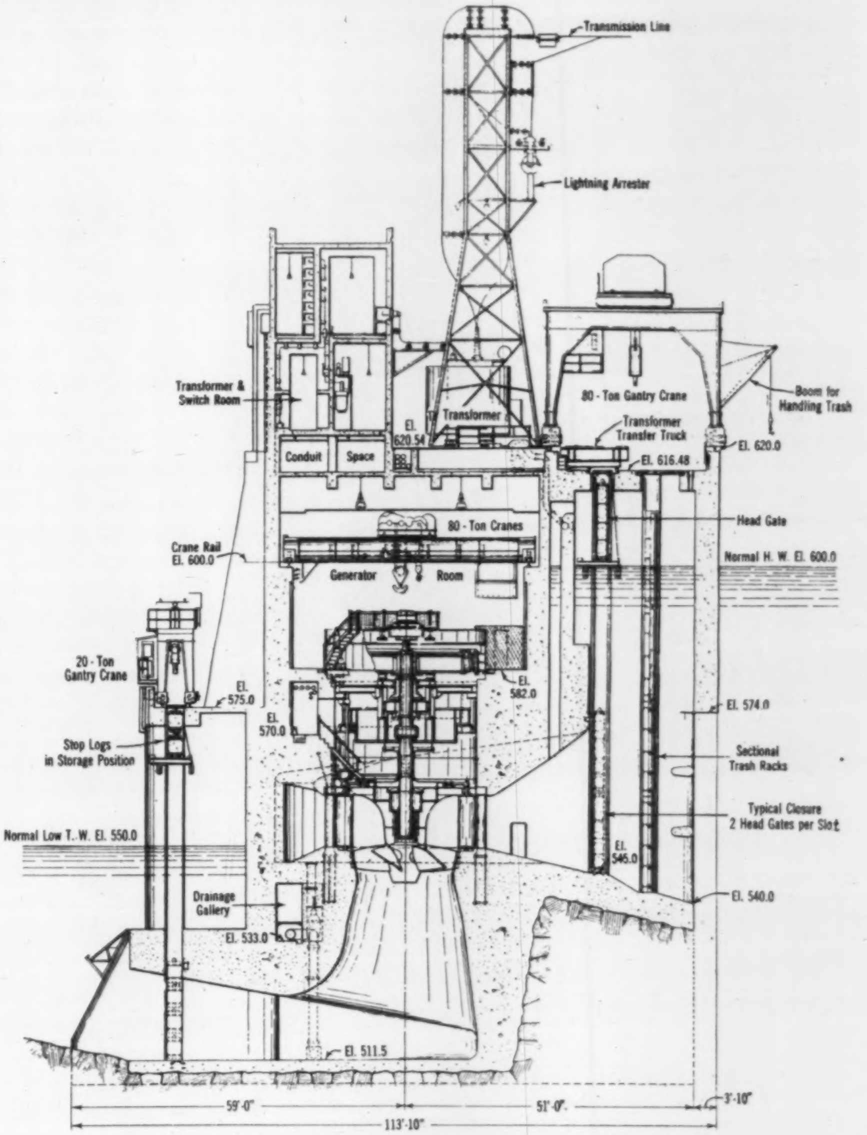
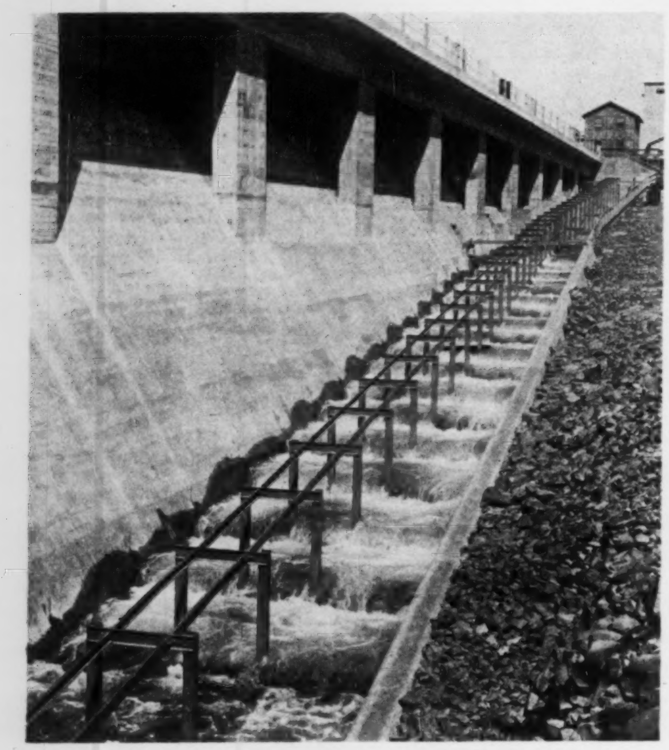

Transmission Line
Lightning Arrester
Transformer & Switch Room
Transformer
El. 620.54
Conduit
Space
80 - Ton Gantry Crane
Transformer Transfer Truck
El. 616.48
Boom for Handling Trash
El. 620.0
Head Gate
80 - Ton Cranes
Crane Rail El. 600.0
Generator
Room
Normal H. W. El. 600.0
20 - Ton Gantry Crane
El. 575.0
El. 570.0
El. 582.0
El. 574.0
Stop Logs in Storage Position
Sectional Trash Racks
Typical Closure 2 Head Gates per Slot
Normal Low T. W. El. 550.0
El. 546.0
Drainage Gallery
El. 533.0
El. 540.0
El. 511.5
59'-0"
51'-0"
3'-10"
113'-10"

Fig. 1. Rock Island Hydro-Electric Development
Cross Section, Main Unit

Next to the power house on the south is the spillway, which will consist of 18 deep gates with a sill at elevation 559, and 19 shallow gates with a sill at elevation 581.5. During the initial development, only the lower half of the deep gates will be installed The normal operating pond level will be maintained by the 581.5-ft. crest. When the seventh unit is constructed, all the gate piers will be extended and the gate structures will be brought up to the ultimate height.

The Rock Island Plant has a low head, ranging from 20 to 51 ft. As the stream flow assumes flood proportions, the operating head will decrease. When the flow reaches 530,000 sec-ft., it may be necessary to further reduce the head by opening the gates in order to avoid excessive backwater upstream. Such flows, which occur about once in three years, are of short duration.

Work on the west cofferdams was begun the latter part of February 1931. Both the upstream and downstream sections were completed by April 15, when they were topped by the flood waters. The work of unwatering the cofferdam in the west channel will not be carried out until after the summer floods have receded, about September 15. After the unwatering, the next step will be the construction of the west spillway and the south abutment. This will complete the major construction work on the plant.



FISHWAY AT THE NORTH ABUTMENT
Fishways Are to Be Provided at Each Abutment

VERTICAL-SHAFT WHEELS TO BE USED

Initially, the power house will be equipped with four 21,000-hp. propeller-type vertical-shaft water wheels. Above each wheel, on the same shaft, will be the revolving part of the generator. The entire revolving unit will be suspended from a vertical-thrust bearing supported on girders placed just below the floor of the generator room. The runner is 18 ft. $9^1/_8$ in. in diameter and consists of six blades attached to a central hub, the whole resembling a ship's propeller. Because of the large variation in operating heads, it was advisable to provide a runner with adjustable blades in order to obtain maximum power throughout the head range. These blades are adjusted by means of a small shaft operating through the main shaft to the hub supporting the blades. A train of gears within the hub provides the proper movement to each blade. A cross section through one of the units is shown in Fig. 1.

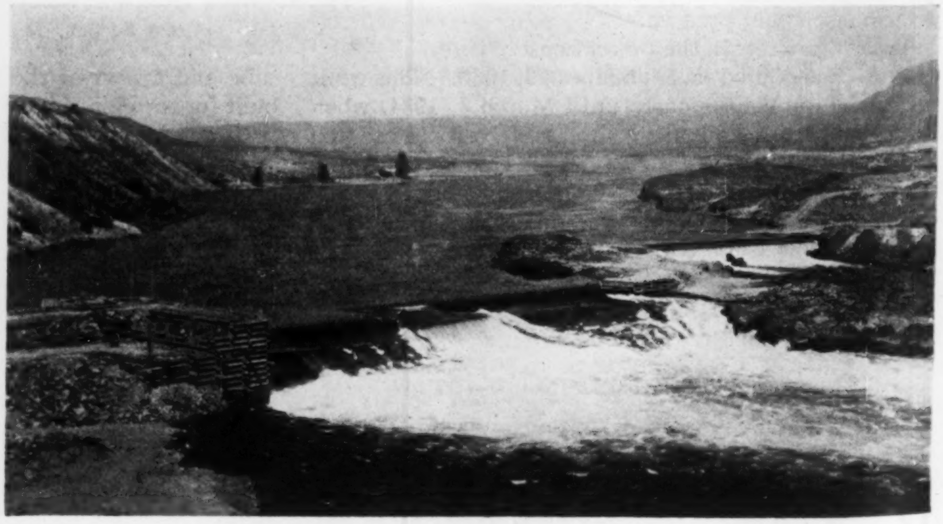
The water wheels are set in reinforced concrete scroll cases, each of which has three head-gate openings 15 ft. 4 in. wide and 27 ft. 11 in. high. The draft tubes are of the modified elbow type. The units will operate at 100 r.p.m. Each water wheel will use approximately 4,500 sec-ft. of water at the maximum head and rated load.

The generators, which are of the umbrella type, have each a rated capacity of 16,667 kva., at 13,800 volts. There are 72 poles in each unit. The weight of an entire machine is 532,000 lb., and that of the heaviest individual piece, 253,000 lb. The stator has an over-all diameter of 28 ft. 8 in. An enclosed recirculating ventilating system, with air coolers, is provided for each main generator.

Power from the Rock Island plant will be delivered at 110,000 volts to the Beverly Park Substation of the Puget Sound Power and Light Company. The transmission line conductor is supported on H-type structures of western cedar with fir cross-arms. As a rule, Western installations have been for high heads, but the Rock Island development is distinctly a low-head plant, and in its type and proportions compares favorably with the Conowingo and Osage plants recently constructed.

The element of construction time is an interesting item. It is now expected that the first two units of the Rock Island Plant, which were begun on January 14, 1930, will be in preliminary service by November 1931. This is truly a fast schedule, considering the various uncertainties of weather, river flow, labor, and other such items. The entire plant will be turned over to the client by August 1, 1932. A maximum of 2,565 men have been employed on this construction.

In an average year, the initial development of 60,000 kw. will furnish 480,000,000 kw-hr., and the ultimate development of 150,000 kw. will provide 1,220,000,000 kw-hr. The development is being designed and constructed by the Stone and Webster Engineering Corporation for the Washington Electric Company, a subsidiary of the Puget Sound Power and Light Company.



UPSTREAM COFFERDAM, WEST CHANNEL
Rock Island Development

Ariel Dam on the Lewis River

By Lyman Griswold

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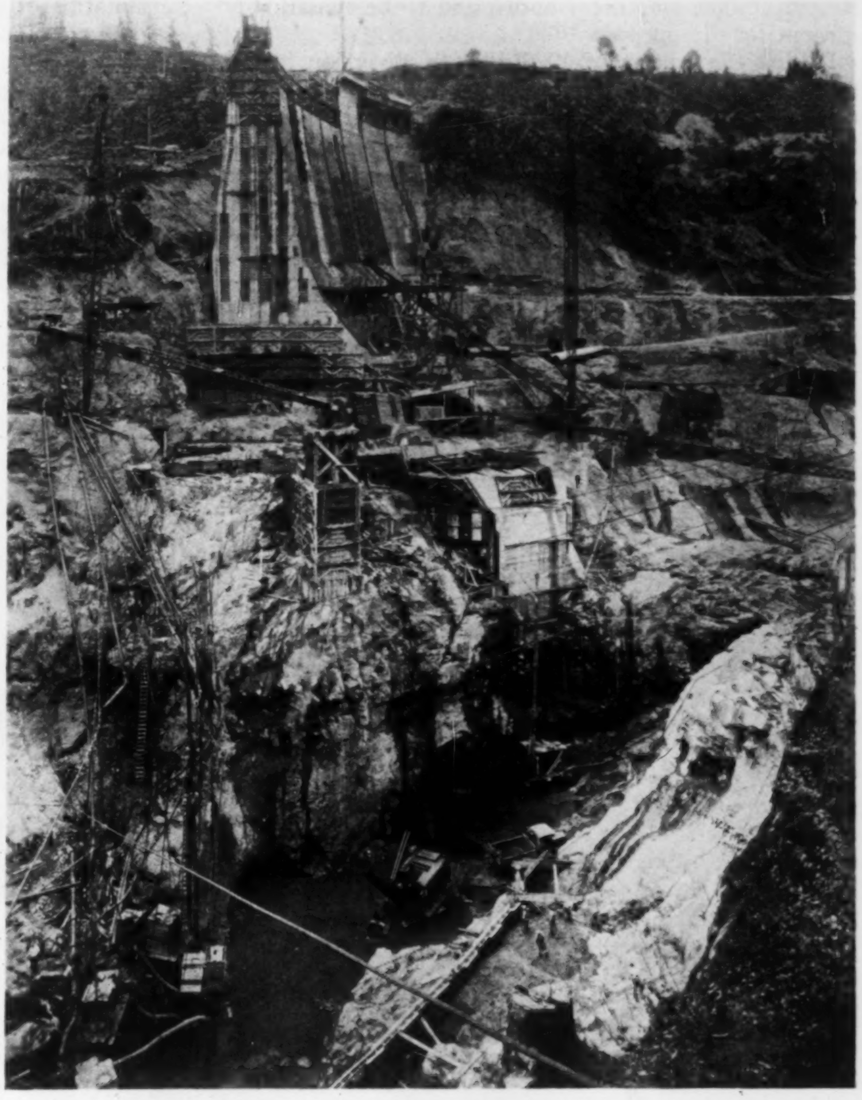
The Ariel Hyro-Electric Development of the Inland Power and Light Company on the Lewis River, Washington, is the first of a series of projects designed to utilize the waters of this river. The height of the dam and the amount of machinery to be installed were governed largely by conditions which will exist when other plants are constructed and the general plan of development is carried out.

Studies of the river, begun in 1911, were for ten years confined to the measurement of stream flow. In 1921 intensive investigations were instituted under the direction of L. T. Merwin, Vice-President and General Manager of the Northwestern Electric Company, which will operate the Ariel Plant under lease. These researches involved the installation of a number of Stevens water-stage recorders on the river and its principal tributaries, a study of the hydrology of the stream based on the records of the recorders, extensive geological investigations at some of the principal sites, topographical surveys by ground methods, and a complete photo-topographical survey of the valley by the aerial method.

The Lewis River has an average annual flow of about 5,000 sec-ft., the low-water flow varying from 700 to 900 sec-ft. for three months of the year, with occasional peak floods during the rainy season, ranging from 50,000 to more than 60,000 sec-ft. These conditions, which had been observed over a period of 20 years, indicated clearly that the working season in the river bed would of necessity be very short, and that, considering the great amount of work to be done, unusually elaborate preparations would be necessary to insure completion within the desired period of less than two years.

Hydrological studies showed clearly that the river could not be diverted from its channel before June 1, and that floods of major proportions could be expected not later than November 1. This meant that there were available between floods about 150 days in which to excavate for the foundation to a point 73 ft. below sea level and 123 ft. below the normal water level, and to bring the concrete in the dam to a point high enough to insure against serious interference by floods. The work involved the excavation of 127,000 cu. yd. of materials and the pouring of 40,000 cu. yd. of concrete from elevation −73 to +50. This feat was successfully performed on schedule time. The bottom of the excavation was reached on September 15, 1930, and the first concrete in the river bed was placed the same evening. On October 1 the concrete had been carried to elevation +50, and the project was safe from serious damage by flood.

Perhaps the most interesting single feature of the development is the geological investigations which were made prior to the beginning of actual construction. In 1924, test pits were dug on land and wash borings were conducted in the river, in order to ascertain the depth of the gorge and any difficulties presented by foundation conditions. As a result, a cost estimate was prepared which has been borne out by the records of the actual construction. Geological investigations previously made



Ariel Dam Under Construction on the Lewis River
River Excavation Down to Elevation −50

had shown this development to be one presenting many difficulties, and for a time serious thought was given to the choice of a site farther upstream, where geological conditions were more favorable. Partly because of its nearness to Portland, but more particularly because of the favorable weather conditions obtaining during the dry part of the present weather cycle, it was decided to build the Ariel Development first. While these investigations were more elaborate and more extensive than in any other case known to us, we feel that the effort and cost were amply justified. In the event of further developments by us, equally elaborate and extensive geological studies will be made.

EXTENSIVE STUDIES CARRIED OUT

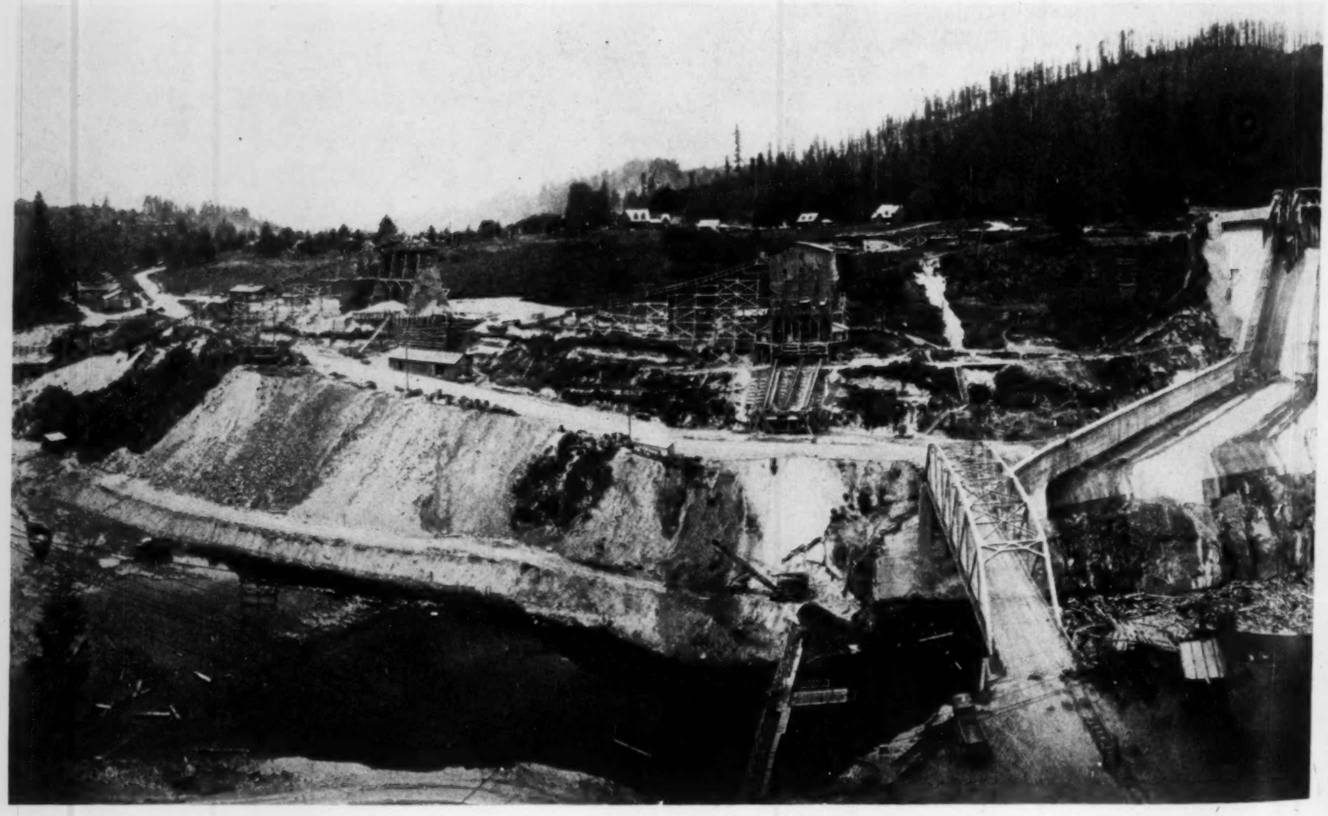
In the course of these studies, some 26,000 ft. of wash borings and diamond drill holes were made, 3,000 ft. of these in overburden and materials from which cores could not be taken. The remaining 23,000 ft. of core were removed and carefully analyzed by Ira A. Williams, Consulting Geologist, of Portland. In addition to the investigations made with drills, the Schlumberger electrical process of determining the depth of rock below the ground surface was used, and found to be remarkably efficient for the purpose.

During the winter of 1929–1930, a diversion tunnel 1,462 ft. long was driven through the hill on the south side of the river. As driven, the bore was 27 ft. wide and 27 ft. high, and had vertical sides to the spring line, a flat bottom, and practically a semicircular arch roof. With the exception of a short portion of the roof at the downstream end, it was lined with concrete, the purpose being solely to increase its capacity. Summer floods are a rare occurrence and seldom equal 10,000 sec-ft. The tunnel was designed, however, to discharge about 16,000 sec-ft. before the cofferdams would be topped and the foundation excavation filled. Rock-filled crib cofferdams were placed above and below the site. Unwatering was performed by electrically driven pumps having a maximum capacity of about 16,000 gal. per min.

The dam has an over-all length of 1,300 ft., which includes an arch 750 ft. in length and a gravity section, including a thrust block 550 ft. long. The arch is of the thin type, the radii varying from 157 ft. to 396 ft. The tops of the parapets are at elevation 240, and the lowest part of the dam at elevation −73. The over-all maximum height is 313 ft. The top width is 26 ft., which includes a small overhang on the reservoir side and one of 6 ft. downstream. At elevation 220 the thickness is about 20 ft. This thickness is not uniform but varies throughout the length of the arch, being in general thicker at the abutments than at the center.

The south, or Clark County abutment rests against a massive cliff. On the north, or Cowlitz County side, it was necessary to construct a heavy gravity block because of the unsymmetrical shape of the canyon. The continuation of this block includes the spillway section, where four Tainter gates, each 39 ft. long and 30 ft. high, and one Tainter gate 10 ft. long and 30 ft. high, were installed. These gates have a maximum discharge capacity of 129,000 sec-ft.

In general, the dam was constructed in sections or blocks, each 30 ft. in length, adjacent blocks being separated by slots 2 ft. wide. These slots were left for the purpose of increasing the rate of dissipation of heat in order that a satisfactory temperature drop might be obtained before closing. In the river section, the slots



GENERAL VIEW OF THE ARIEL

were carried down to sea level, and below this elevation an ordinary construction joint was formed, with provision for grouting. In most of the blocks the temperature drop was satisfactory without artificial cooling, but where sufficient time was not available for the loss of heat by natural means, ducts were cast in the blocks and artificial cooling by the circulation of water was resorted to. The result of this method was satisfactory.

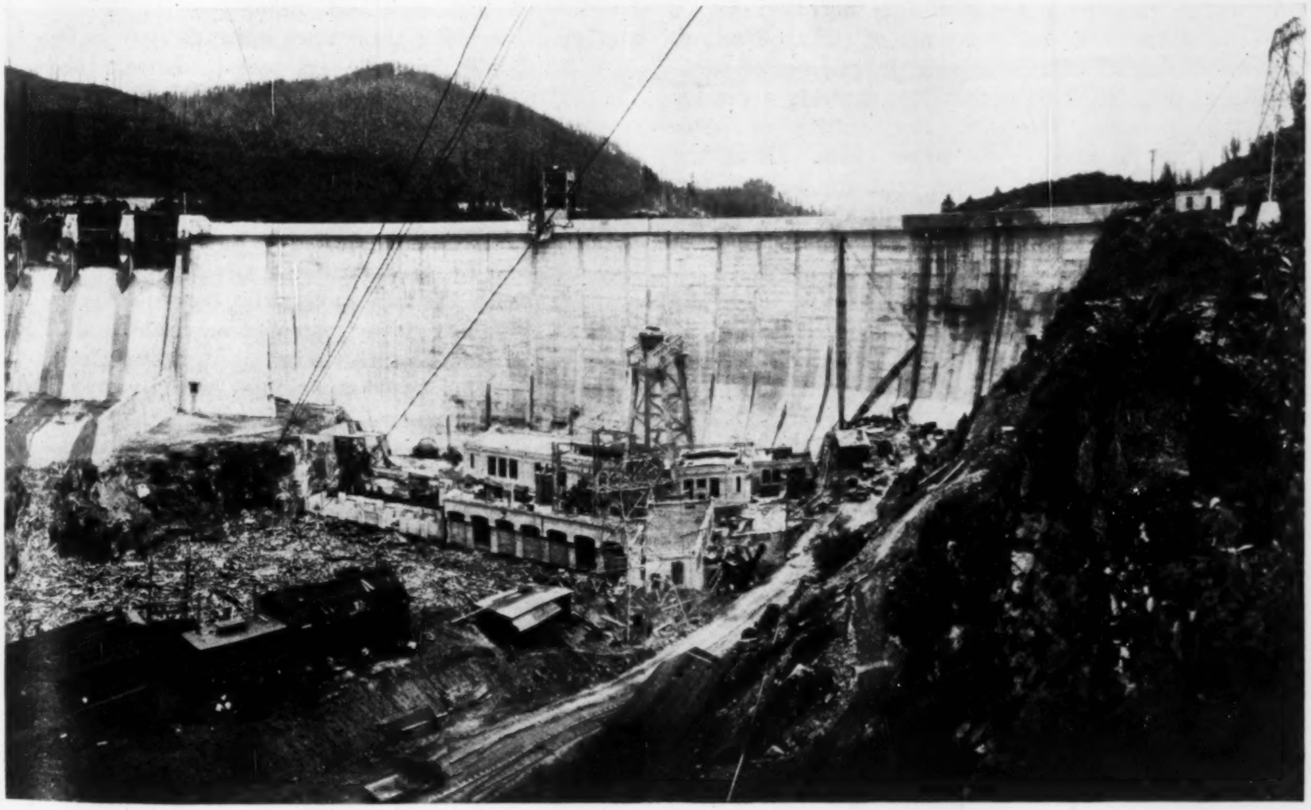
No copper water stops were used in the arch. In the gravity block, which was also constructed in 30-ft. sections, but without slots, the usual copper water stops were provided and the joints were grouted just before the diversion tunnel was closed. An inspection gallery is located 10 ft. downstream from the upstream face of the dam, and extends throughout the gravity block. It is provided with the usual vertical drains leading to the top of the dam and with the customary relief pipes extending into the rock foundation. When completed, the foundation below the gravity block was thoroughly grouted.

Coarse and fine aggregate were obtained from the river bed. A part came from the foundation excavation, and the balance and major portion from a sand and gravel bar a few hundred feet downstream. Excavation was performed with dragline buckets. The material was separated into three grades—sand, pebbles, and cobblestones. Material less than $^1/_4$ in. in diameter was classified as sand; that from $^1/_4$ in. to $1^1/_4$ in. in diameter, as pebbles; and that from $1^1/_4$ in. to 3 in., as cobblestones. After these materials were separated, graded, and washed, they were deposited in separate stock piles. It is an interesting and very unusual fact that more than 300,000 cu. yd. of concrete had been placed before there was a cubic yard of excess of any of the three, so nearly perfect had been the work of nature in proportioning the materials.

The power house, which is immediately downstream from the dam, extends directly across the river bed and is supported by an arch which spans the old river channel. Provision was made for four units, each of 45,000-kw. capacity, but only one of these is being installed at present. The future load factor of this plant will probably be 35 per cent. It is made possible and profitable by the exceptional storage possibilities of the basin.

A reservoir with a surface area of about 4,000 acres will be created. The plant is designed for a maximum draw-down of 65 ft., and if this is obtained 220,000 acre-ft. of storage will be available. However, such a draw-down is not contemplated. The present operating schedule provides for a normal draw-down of about 10 ft., with a maximum of 15 or 20 ft. at rare intervals.

The project was designed in the offices of the Electric Bond and Share Company, under the direction of A. C. Clogher, M. Am. Soc. C.E., Chief Hydraulic Engineer. It was constructed by the Phoenix Utility Company, of which T. C. Wescott was General Manager, and H. F. Lincoln, Project Manager, in charge at the site. The Federal Power Commission was represented by Phil Dater; and the State of Washington by Charles J. Bartholet, Supervisor of Hydraulics, and by D. C. Henny, M. Am. Soc. C.E., Consulting Engineer. For the interested companies, Guy W. Talbot, President of the Inland Power and Light Company; and of the Northwestern Electric Company; and L. T. Merwin, Vice-President and General Manager of the Northwestern Electric Company, were in general charge. I served the Northwestern Electric Company and the Inland Power and Light Company as Consulting Engineer



Dam and Appurtenant Structures

Factors in Successful Highway Operation

Modern Traffic Requires Economic Roadway Design and Location and a Protected Right-of-Way

FORMERLY highway construction was largely a matter of improving the surface of existing horse-and-buggy routes. But now that there are in the United States more than eight automobiles for each mile of improved highway, the economics of highway operation has become an important field of engineering study. At the Tacoma Convention, on July 9, the City Planning and Highway Divisions were fortunate in hearing up-to-the-minute discussions of various outstanding elements in the successful design and operation of our highways, looking toward a safer and more efficient movement of traffic.

Investigations directed by William J. Fox in Los Angeles County have indicated that there is a straight-line relationship between a given population and the area of the highways which can economically serve it. As Chief Engineer of the Automobile Club of Southern California, E. E. East has found that the traffic on through routes and the communities along such routes are both better served by the by-passing of traffic around these centers. It is pointed out by L. I. Hewes that highway engineers can profit by the application of some of the principles of railroad operation to the regulation and control of highway traffic. Especially should advertising signboards be barred from the highway right-of-way.

In its progress report, the Committee on Street Thoroughfares Manual of the City Planning Division submits for consideration a set of definitions of terms for various types of thoroughfares.

Population Density Related to Roadway Area

By William J. Fox

Associate Member American Society of Civil Engineers
Chief Engineer, Los Angeles County Regional Planning Commission

THE present advanced development of the highway is the direct result of the increased use of the automobile as a means of transportation. California leads all other states in the number of automobiles in proportion to population, showing a car for each 2.64 persons. However, the density of automobiles is not confined to the larger cities. Of all the cars in the United States, 57 per cent are registered in towns having populations of less than 10,000. This country has 76.3 per cent of all the automobiles in existence.

This extensive use of the automobile is reflected in the mileage of improved highways. The United States has a total of 3,016,000 miles of road, of which 666,000 miles, or 22.8 per cent, are surfaced and improved. Taking the country as a whole, this gives a ratio of 8.17 automobiles to one mile of road. In California the ratio is 79 automobiles per mile of surfaced road, or one to every 67 ft. The population per automobile in California, in the United States as a whole, and in four other countries, may be seen from the following tabulation:

Mexico	202	Canada	8
Germany	102	United States	4.6
France	33	California	2.64

Automobiles have a definite relationship to population. Experiments indicate that there is an equally definite relationship between the number of automobiles in a given area and the amount of paved roadway required for the safe and efficient movement of traffic on through highways and major streets. Results of studies made along these lines are indicated in Fig. 1. As is usual in highway design, peak loads were taken as the controlling factor in working out these diagrams.

Any community's requirement as to paved street area is a function of the number of automobiles operating in the district. The use of the street pavement graph, shown in Fig. 1, is of inestimable value for the following purposes: (1) to check existing paved highways in a city, in order to ascertain if there is sufficient roadway space for safe and efficient traffic movement; (2) to design a through highway system commensurate with the predicted population; (3) to set up a program of highway construction based upon five- or ten-year periods of population increase.

Obviously, the use of such a graph is a distinct departure from the "guess and trial" method of designing a future highway system. If the proportion of travel in a given direction and an estimate of the ultimate population of a community are known, the graph provides an accurate means of designing the highway system in a manner consistent with the natural demand. This chart was compiled after an exhaustive study of existing conditions in the 45 cities in Los Angeles County. While the dimensions and general mechanical performance of cars may change through the years, such changes or improvements will not appreciably affect the amount of paved through highways required for a given number of automobiles.

In the process of preparing comprehensive highway plans for the County of Los Angeles, the Regional Planning Commission has endeavored to reduce to a scientific basis the formulas for determining the frequency, direction, and width of roads required in the system to serve ultimate traffic needs. The prevailing practice in preparing plans for future road systems, as to frequency and width, has been generally predicated upon a twofold basis: first, the simple judgment of the persons preparing the plans; and second, traffic counts and traffic distribution studies. The fact that opinion and judgment will vary considerably in a matter of this kind needs no discussion.

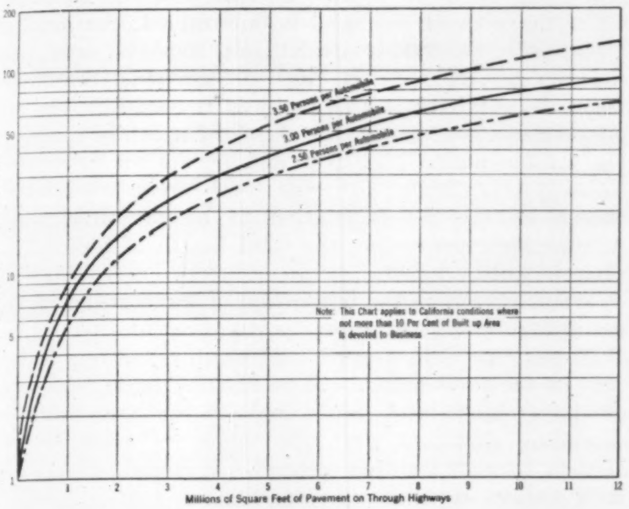


Fig. 1. Street Area Graph

By the Regional Planning Commission of Los Angeles County

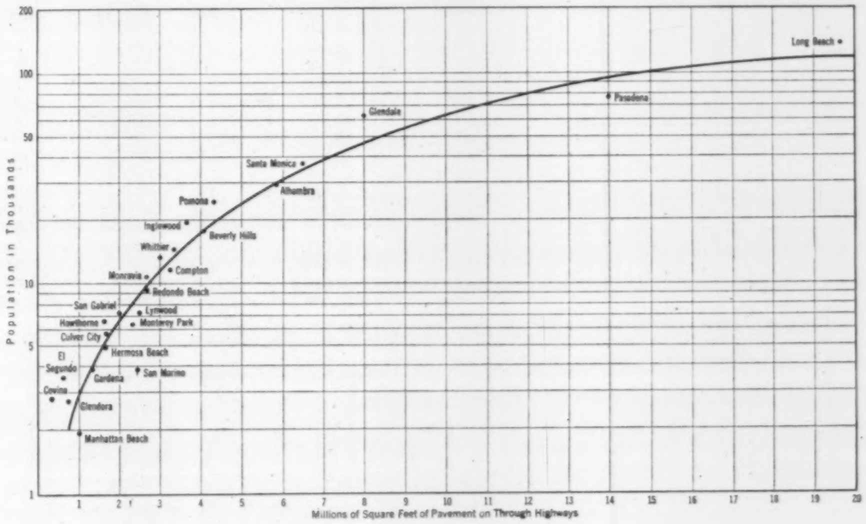


Fig. 2. Relation of Area of Through Streets to Population

For 45 Cities of Los Angeles County

Traffic-load diagrams, showing the origin, destination, and distribution of traffic, are extremely helpful in preparing a comprehensive highway plan, particularly with regard to the type of pavement to be used and the time when a particular road should be constructed. However, such information has doubtful value in the preparation of a comprehensive plan for ultimate number, width, and frequency. The reason is that traffic-load diagrams cannot reflect conditions which will exist when a region is fully developed—they simply show the traffic conditions of the moment, under a given set of circumstances.

After several years of experimentation with various methods of predicting the future demand for highways as to frequency, width, and direction, we have concluded that the most valuable factor to employ in such a determination is that of ultimate population, used as a load factor. In order to arrive at a more or less scientific method of preparing a comprehensive highway plan, charts showing the trend of the population load should be employed.

The direct relationship between the number of automobiles in a given area and the population in the same area might present a true basis for determining the number of main streets per mile that should be laid out by the community for traffic purposes. If the population at a given time, for a designated area, is P_1, and the ultimate population for the same area is P_2, we start with the assumption that $(SA)_1$, the existing square footage of through streets used by the traffic caused by P_1, is in proportion to $(SA)_2$, the square footage required by the ultimate population, P_2. When the ultimate population, or P_2, which the cities are being designed to accommodate, has been determined, $(SA)_2$, the amount of pavement required for the ultimate population, can be estimated by the simple proportion:

$$\frac{P_1}{P_2} = \frac{(SA)_1}{(SA)_2}$$

In the case of the cities in Los Angeles County, the ultimate population was calculated with the aid of zoning maps, where available, by assigning 15 people to the acre for districts made up of one-family dwellings, 80 people to the acre for districts containing apartments and multiple-family houses, and 50 people to the acre for ordinary commercial districts. On an average, this gave a population of about 15,000 per sq. mile, all uses included. These factors will vary in different parts of the country. Those used in this instance are peculiar to southern California, and studies over a period of time have demonstrated their consistency.

In order to ascertain the degree to which this more or less empirical formula applies to the development in this part of California, a survey was made in each of the 45 incorporated cities in the County of Los Angeles, the results of which are plotted in Fig. 2. These cities vary in population from less than 1,000 in the case of West Covina, to more than 1,000,000 in the case of Los Angeles. Data for 17 of them are given in Table I. The survey revealed a striking consistency in the ratio of population, number of automobiles per 1,000 persons, and amount of square feet of road surfacing used in each city per 1,000 population. This relationship is more striking when it is realized that the cities studied vary in character—they include beach communities, agricultural centers, industrial districts, and heavy commercial centers.

Undoubtedly there are factors which tend to modify these influences, such as the amount of business in a given city and the relationship of the city to through traffic between a beach community and a distant residential center. While further study of the extent of these influences will be interesting, it seems that their effect upon the amount of road surface is negligible. The greatest divergence from the ratios which appear to prevail in the 45 cities under study seems to result when the area of a city is large in proportion to its present population and when it has been necessary, as a result of jurisdictional conditions, to carry roadways to the boundaries of the city in order to connect them with those of an adjoining town or with the county highway system.

In a few remote cases such a condition has caused communities to construct an abnormal amount of road work. There are also instances in which communities have, in the promotion of untimely subdivision development, constructed roads far in advance of their needs. While these roads were in the proper location for main highways, they were not considered in the study because they were not used by through traffic.

Clearly there exists a true relationship between the population and street area required for through traffic arteries—or, what is the same thing, for the number of arterial highways per mile, distributively speaking. This fact was borne out by the investigation, which indicated that the cities with a population of 20,000 had a total street area devoted to through traffic double that of a city of 10,000, while those of 80,000 population in turn had twice as much as those of 40,000.

In applying the street area graph, the procedure is similar to that of determining the amount of steel in a reinforced concrete beam. In the case of a beam, when the area of the steel is determined, the area of $^{1}/_{2}$-in. or $^{1}/_{4}$-in. rods is divided into the total area, and the number of rods is thus determined. Similarly, when the total street area required is found by the use of the street area graph, the standard width of roadway (74 ft. for major highways and 56 ft. for secondary highways) is divided into the total street area, which corresponds to the population to be accommodated. A quantity representing the total length of highway is thus obtained.

Traffic checks on the proportion of travel will give the percentage of distribution, as to direction. That is, if 40 per cent is in a north and south direction, 40 per cent of the total length will be distributed in highways extending north and south, and 60 per cent in those extending east and west. No claim of scientific exactness is made for this method. However, experience with other methods of ascertaining the frequency, direction, and width of highways required to care for the ultimate needs of a particular community, leads me to believe that the street-area graph method, based on population as a load factor, gives results which are reliable and in accord with actual and measurable conditions.

TABLE I. RELATION BETWEEN POPULATION, PAVEMENT AREA, AND NUMBER OF AUTOMOBILES IN THE SAN GABRIEL VALLEY

	AREA (Square miles)	POPULATION		ASSESSED VALUATION	SQ. FT. OF PAVEMENT ON THROUGH STREETS			PERSONS PER AUTOMOBILE
CITY		Present	Ultimate		Present	*	Ultimate (as planned)	
Alhambra	6.20	29,472	100,000	$26,091,000	2,400,000	60	9,720,000	3.50
Arcadia	9.70	5,216	150,000	8,500,000	400,000	20	13,867,000	3.50
Azusa	4.06	4,808	74,000	2,579,000	530,000	30	5,440,000	3.50
Claremont	3.33	2,719	25,000	3,087,000	300,000	20	5,306,000	4.00
Covina	0.86	2,774	13,800	2,213,000	400,000	20	1,600,000	3.50
El Monte	1.14	3,479	18,200	2,233,000	380,000	100	1,929,000	3.50
Glendora	2.18	2,761	35,000	2,099,000	395,000	50	3,357,000	3.50
La Verne	1.25	2,860	18,000	1,685,000	290,000	5	2,321,000	3.75
Monrovia	7.91	10,890	80,000	10,959,000	1,200,000	75	6,348,000	3.50
Monterey Park	5.00	6,406	70,000	4,436,000	680,000	50	5,585,400	3.50
Pasadena	17.70	76,086	300,000	181,791,000	6,000,000	80	24,119,900	3.00
Pomona	12.50	20,804	200,000	16,254,000	2,000,000	50	18,632,000	3.50
San Marino	3.50	3,730	35,000	10,696,000	201,000	30	3,100,000	2.70
San Gabriel	3.00	7.224	48,000	5,698,000	480,000	25	5,453,000	3.75
Sierra Madre	2.94	3,550	47,000	3,758,000	460,000	10	2,467,200	3.50
South Pasadena	3.14	13.730	50.000	14,715,000	1,400,000	25	2,786,800	3.25
West Covina	8.00	769	128,000	2,628,000	40,000	1	8,958,400	3.50

* Figures in this column indicate percentage of present capacity used under peak-load conditions.

By-Passing Traffic Around Small Towns

By E. E. EAST

ASSOCIATE MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
CHIEF ENGINEER, AUTOMOBILE CLUB OF SOUTHERN CALIFORNIA, LOS ANGELES

MANY villages, towns, and cities have had their humble beginnings in that unique commercial and social center of rural America of a quarter of a century or more ago, the cross-roads store. Yet it is not justifiable to attach great importance to every highway intersection in a nation traversed by nearly three million miles of highways. Transportation is essential to the life and growth of a town, but it does not follow that the highway or railway, or both, as the case may be, are the cause of the town.

When Father Junipero Serra arrived at the site of the present City of Los Angeles in 1769, he found a location which met his specifications for a mission village—water, sunshine, and fertile soil. Here he founded the mission of Los Angeles, around which the present city of over one and one-quarter million people has developed. Although the city could not exist without its railroads and highways, the fact remains that these facilities have developed because of Los Angeles and not Los Angeles because of these facilities. This distinction is clear, I believe, to the transportation engineer, but the citizen of the average town or city generally holds the opposite opinion, and this point of view constitutes the major opposition to the by-passing of traffic around small towns.

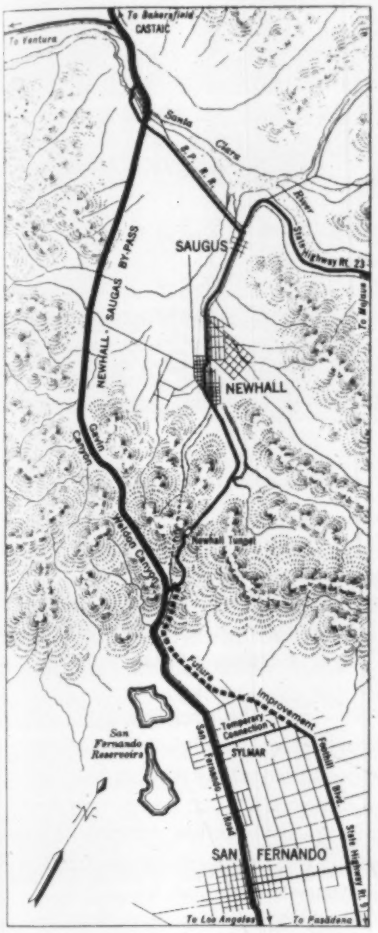

To Bakersfield
CASTAIC
To Ventura
Santa Clara River
S.P. R.R.
SAUGUS
State Highway Rt. 23
To Mojave
NEWHALL - SAUGAS BY-PASS
NEWHALL
Canyon
Newhall Tunnel
Future Improvement
Temporary Connection
San Fernando Reservoirs
SYLMAR
San Fernando Road
Foothill Blvd
State Highway Rt. 9
SAN FERNANDO
To Los Angeles
To Pasadena

Fig. 1. Newhall-Saugus By-Pass

By-passing is one of those intensely human problems, in the analysis of which the basic facts are too frequently lost in a maelstrom of selfish interest, political wire pulling, newspaper comment, and curbstone argument. In principle, by-passing is fundamentally sound for it recognizes as a basic premise the prior right of the public to a maximum enjoyment of and return from a facility constructed from public funds, as against the desire of those who always constitute a minority, and are frequently parasitic, to appropriate such facility to their own personal ends. Observe the travesty in the daily and spectacular parade of motor vehicles from all parts of the country, down any "Main Street," led by the local street car and paying tribute to the fantastic notions of America's small town merchants.

In California, where substantial progress has been made in constructing major highways around the congested business districts of small towns and cities, by-passes have been proposed in each case with one of the following purposes in view: first, shortening distance, correcting alignment, or eliminating hazardous conditions, all of which have to do with efficiency in vehicle operation; or, second, relieving congestion on business thoroughfares, which has to do with both the efficient operation of vehicles, and convenience and efficiency in the transaction of business.

STRONG LOCAL OPPOSITION ENCOUNTERED

In southern California a number of by-passes have been completed which are of unusual interest both because an amazing increase in the efficiency of the highways resulted from them, and because the projects were completed in the face of what at the outset appeared to be insurmountable opposition. One of these is the Newhall-Saugus by-pass (Fig. 1), which is about seven miles long, and is a part of the great commercial highway connecting the City of Los Angeles with the San Joaquin Valley, one of its principal sources of food supply. The project was designed primarily to relieve congestion in a tunnel section through the Newhall Mountains.

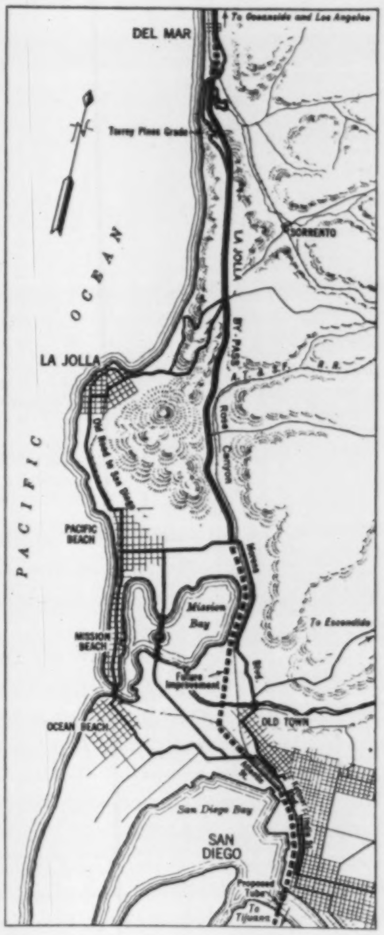

To Oceanside and Los Angeles
DEL MAR
Torrey Pines Grade
PACIFIC OCEAN
LA JOLLA BY-PASS
SORRENTO
A.T.&S.F. R.R.
LA JOLLA
Old Road to San Diego
Rose Canyon
PACIFIC BEACH
Morena Blvd
Mission Bay
To Escondido
MISSION BEACH
Future Improvement
OCEAN BEACH
OLD TOWN
San Diego Bay
SAN DIEGO
Proposed Tube
To Tijuana

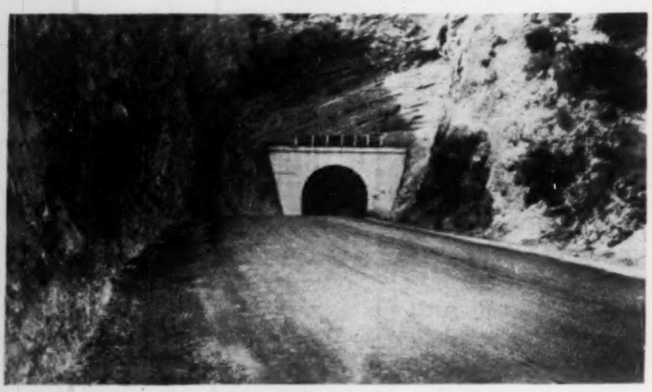
Fig. 2. The La Jolla By-Pass

The two towns of Newhall and Saugus, although not primarily a cause of congestion, were involved in the re-routing of travel and fought the project from every angle. The governor of the state, the legislature, the board of supervisors, the chamber of commerce, and every service club and organization were appealed to in a frantic effort to protect the fancied rights of a few citizens who had "squatted" on the public domain. But every argument presented by the opposition was met with facts that could not be refuted, and the by-pass was built. In effect, it moves a great agricultural area one or more hours closer to one of its largest consuming centers.

The new entrance road to San Diego, frequently referred to as the La Jolla by-pass (Fig 2), is about 22 miles long and will for all time provide the city with a high speed entrance from the north, the direction of its maximum highway travel. The largest and most difficult section of this project has been completed and is now in operation. Intermediate centers are adequately served by the old highway, which has been greatly improved, and they are relieved from the annoyance and congestion caused by commercial and non-purchasing travel on their principal business and residential streets.

One of the interesting angles to the bitter controversy over the construction of this by-pass was what appeared to be the irrelevant viewpoint from which certain individuals and organizations aligned themselves for or against it. One active opponent was found to be quietly promoting a little by-pass of his own, intended to direct travel through his subdivision, and thence by a circuitous path to San Diego.

A phase of this problem, which in a large measure led to the development of the present plan of improvement, was the effect of congested business centers on travel between Los Angeles and San Diego. The road has a length of 134 miles, of which approximately 33 per cent lies in incorporated territory. Because of the congestion on business thoroughfares in this area, approximately 50 per cent of the driving time was spent on 33 per cent of the distance. In effect, San Diego and Los Angeles were being moved farther apart, although the theory on which the highway was financed and constructed was that these two cities and the intervening centers should be brought into closer contact.



The Newhall Tunnel, a Two-Lane Bottleneck
Avoided by the Newhall-Saugus By-Pass

Construction on the Weldon Canyon Cut-Off
On the Newhall-Saugus By-Pass

Prior to the improvements which have been lately carried out, the average driving time from Los Angeles to San Diego was about $5^1/_2$ hours. Today the trip can be made in comfort and safety in about $3^1/_2$ hours. This increase in the efficiency of the road is in some part due to more lenient speed regulations, better automobiles, and general improvement of the highway, but very largely to better facilities for getting around and through congested districts.

The Newhall-Saugus and the La Jolla by-passes are typical cases in which the benefits to traffic are large but so intangible that they cannot be readily measured in terms of dollars and cents. The benefits resulting to property and business in a congested section of a town are also difficult to measure. Owing to the extreme prejudice of the average property owner or business man against by-passing, the true effect of a by-pass on property or business cannot be obtained until a considerable time after its inauguration.

SAVINGS IN DOLLARS AND CENTS

There is another type of by-pass in which the savings to the public, and particularly to the operators of vehicles, can be measured in dollars and cents. One of the principal transcontinental highways passes through a small town on the eastern boundary of California, not because of careful routing, but largely for the reason that, as automobile travel developed over this route, it followed the line of least resistance.

A careful study of possible locations in the vicinity has disclosed a practicable route that can be built at a per mile cost not greater than the per mile cost of improving the present route, and at a saving in distance of 31 miles. If we assume an initial cost of construction of $30,000 per mile, there is a saving in this item alone of $930,000. The annual interest on this sum, at 5 per cent, would be $46,500. Assuming maintenance at $400 per year per mile, the saving is $12,400 a year. The total saving in what might be termed fixed charges is $58,900 per annum.

Assuming the cost of operating a motor vehicle at 10 cents a mile, there is a saving of $3.10 per car and, for the 200 through vehicles daily now using this route, $620 a day, or $226,300 a year. The total annual saving is the appreciable sum of $285,200. Here, in dollars, is the picture from the traffic viewpoint without any attempt being made to place a value on the time saved by motorists.

It is of interest, also, to study the problem as it affects the opposition. The town has a population of 3,140 people, of whom 752 are employed by the railway company. Assuming two dependents to each employee, there are 2,256 who are in no appreciable manner affected by the alternate routing. Of the remainder, there are some 250, including dependents, who are engaged in occupations related to railroad operation. Another 500 or more are engaged in businesses incident to a town of this size, leaving less than 150 people or, we will say, 50 wage earners and their dependents who may be assumed for purposes of analysis to receive their livelihood in part or totally from automobile travel.

It is extremely difficult to determine the average tourist expenditure per car day. Our organization has from time to time collected information relative to this subject, and from the best sources available a daily expenditure of $5.25 per car has been assumed. Hotels, camp grounds, filling stations, and restaurants are found at less than ten-mile intervals along this road, and this fact, supplemented by a certain amount of study, leads to the conclusion that, of this average expenditure, not more than $1 is spent in the town under consideration. This means an average amount of $4 daily for each individual engaged in serving this highway travel. It is obvious that the resulting profit would not be sufficient to induce these people to remain in business in this section if this were their only source of revenue.

The insistent demand of the small town business man that all highway travel pass along the principal business thoroughfare is, in large measure, due to a feeling that travel by motor car is essentially a sight-seeing occupation; that motorists as a class have no place in particular to go; and that all that is necessary to get them to put up at the "New Central," lunch at the "White

Front," or buy a lot in the newest subdivision, is to herd them down "Main Street." Highway and traffic engineers have nourished this idea rather than discouraged it, through a failure to analyze highway traffic. Traffic checks designed to acquaint the average citizen with the habits of the American motorist and the character of travel entering any given town or city, are of inestimable value in overcoming many of the false ideas which have developed around one of our great industries, that of highway transportation.

For a town of appreciable size, our studies have developed the following facts, which in most instances may be taken as the basis for studies having to do with by-passing any particular city or town. First, by far the greater number of all vehicles entering or leaving any given town or city are local. Second, these local vehicles are entering the town or city for the purpose of transacting business, or as their destination. Third, this local travel entering and leaving the city, together with vehicles operating within the city boundaries, represents the automobile purchasing travel, and as such should be afforded a maximum of convenience. Fourth, the remainder of the motorists have a destination in view, and forcing them down "Main Street" inconveniences purchasing travel, to the ultimate loss of business on the street.

Our studies concerning highway efficiency, which cover a period of approximately ten years, have developed some interesting facts. In the county of Los Angeles about 846,000 motor vehicles are registered. For purposes of traffic analysis the county has been divided into five concentric areas. Over a period of several years we have collected information as to the movement of motor vehicles over these several boundaries. This movement during the year 1930, expressed in number of vehicles daily, is in round numbers as follows: in and out of the county, 75,900; in and out of the metropolitan area, 305,900; in and out of the residential area, 705,700; in and out of the outer congested area, 618,700; in and out of the central business area, 531,500. Cars entering the county represent about 11 per cent of those entering the residential area.

Ventura County, joining Los Angeles County on the west, has a motor vehicle registration of about 22,000. An analysis of traffic entering and leaving the City of Ventura, the county seat of government and the principal city of the county, shows 80 per cent originating in the county and 20 per cent outside the county. Of all vehicles entering within a given period, approximately 80 per cent were destined for the City of Ventura.

Through travel is about 20 per cent of the total. Of this class 67 per cent passed through the city without stopping, and the remainder, which stopped chiefly for gasoline, could have received, and in most cases did receive, this service before or after reaching the congested section of Main Street. Similar studies in other cities and towns located on through highways show about the same relation between local and through travel. In all instances the behavior of through travel is about the same, that is, the great majority passes through towns without stopping.

Type of Business Attracted by Through Traffic
In the Town of Saugus

In California, the establishment of the principle of by-passing small towns and cities has been exceedingly difficult, but through persistent and consistent effort substantial progress has been made. Yet there are many towns and cities where even the most enthusiastic advocate of by-passing would not at this time suggest such action because the density of traffic on the principal thoroughfares has not become a serious problem either to traffic or to business. When congestion reaches the point where either travel or business, or both, suffer, by-passing will be the solution.

Increasing Highway Efficiency

By L. I. Hewes

Member American Society of Civil Engineers
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With the developments of the past three decades in the field of highway design, construction, and maintenance, the problem of highway operation has assumed increasing importance. To a considerable extent, the solution of this problem has devolved upon individuals and organizations outside the engineering profession. Nevertheless, there has been a growing consciousness on the part of highway engineers of the requirements of successful operation. The time when the highway designer was primarily interested in construction features has passed.

In approaching this subject of highway operation, it is desirable to visualize highway traffic as a whole. The mass movement of our 26,500,000 motor vehicles, operated largely on 500,000 miles of primary highways, is made up of similar details in each locality. The annual death toll, which exceeds 30,000, is not confined to particular zones or regions. Accidents are distributed throughout the country, and are closely related to problems of local highway operation.

Traffic has at present increased beyond public plans and facilities for its control. In studying the problem that it presents, I believe there is a useful analogy between highway and railroad operation. Highway engineers must frankly acknowledge their indebtedness to railroad engineering methods for the beginning of our modern highway development. But have they not somewhat neglected the useful experience of railroad operation in planning highway operation? I refer specifically to the railroad right-of-way. Within their rights-of-way railroads are absolutely supreme, and the scheduled operation of trains is so completely controlled that there results a progressive increase in the safety of railroad travel. It is impossible for the individual owner of abutting property to obtain ingress to or egress from a railroad right-of-way. Only on the grounds of necessity and public convenience can access be obtained.

GILLIO BROS.

OBSTRUCTED VIEW ON CURVE
Billboard Advertising Creates Traffic Hazard

This is in emphatic contrast to existing highway conditions. The matter of parking automobiles should, for instance, be investigated. It is not a question of how long an individual car may park, or of how many free zones there may be for such parking. The larger question in our older cities is the width between curbs and the relation of this width to the commercial activity that develops constant parking.

Highway designers have been increasingly aware of the part that their work plays in the creation of better operating conditions, and their recent achievements in this direction have been many. The struggle for the elimination of grade crossings is a separate problem. In spite of the sacredness of the railroad right-of-way, highway traffic constantly succeeds in increasing the number of new crossings. This increase, however, is largely on the local highways. On the main routes grade-crossing elimination is progressing rapidly in spite of its high cost. There were 956 eliminations by grade separations and 3,547 by relocation from January 1, 1917, to June 30, 1929, on the Federal Highway System by Federal Aid, exclusive of all eliminations independently by the states.

In addition to the built-in, or design, features for safety in operation, the highway engineer has in recent years introduced such devices as the traffic strip, the multiple traffic lane, snow fences, and, in cooperation with the railroads, warning devices and signals at railroad crossings. In some states highway equipment has been painted a brilliant red, white, or yellow to warn the motorist. Dust has been eliminated as an element of danger in operation, and increased attention has been given to detours where roads are under construction or repair. In a further effort to increase safety in highway operation, the American Association of State Highway Officials and the U.S. Bureau of Public Roads have adopted standard numbering, warning, and directing signs throughout the Federal highway system.

HIGHWAY RIGHT-OF-WAY NOT RESPECTED

In the past, in great contrast to the sacredness of the railroad right-of-way, the highway right-of-way has been the field for more or less indiscriminate outdoor advertising and other encroachments. In the days of the horse-drawn vehicle, before motor traffic was in sufficient volume to especially attract the advertiser, casual signs on rocks, trees, and fences, and even the more substantial bulletin-board type of sign, were generally tolerated. With the increase in the volume of traffic, however, and the corresponding multiplication of signs within the right-of-way, local, state, and Government authorities have been forced to consider measures for the removal of such highway parasites. In fact, highway departments in all the states have made vigorous campaigns for the removal of all signs actually within the right-of-way.

Recently the states of Wisconsin, Pennsylvania, Oregon, and California legislated for the removal of all signs outside the right-of-way which were illegally placed on private property. The success of this campaign, which began in May of this year, is indicated by press reports stating the removal by Wisconsin of 52,000 signs, by Pennsylvania of 32,000, and by California of approximately 50,000. The country-wide movement against highway advertising has raised in the minds of advertisers a question as to the value of such advertising.

It is unfortunate that organized outdoor advertising has grown to the extent that it involves property rights in organized companies, but it must be remembered that this method was developed without permission from the authorities or from the motorists who pay for and operate the highways. Perhaps, in the beginning, the advertisers themselves did not realize the extent to which their business would interfere with the pleasant and safe operation of highways.

It is obvious that, in the operation of railroads, any confusion of signals, semaphores, warning lights, or other precautionary devices resulting from advertising signs in the line of vision would be intolerable. However, equally adverse conditions prevail intermittently throughout the vast mileage of American highways. For instance, the adoption of the uniform numbering, directing, and warning signs on the Federal highway system will not increase the safety of highway operation if these devices are interfered with by a background of mass advertising, which blurs and confuses the vision of the motorist. The warning signs are usually distinct and well placed, but they do not catch the eye of the driver as easily and quickly as they should.

California is adopting at railroad crossings a 24-in. red Neon sign visible a mile away and legible at several hundred feet—an excellent device, but useless if there is too much competitive illumination. The situation is particularly serious because of the fact that direction and warning signs are most needed at intersections, cross roads, and near railroad crossings, where the adverse condition of the advertising background is emphasized. So conditions that would outrage railroad operators obtain within a vastly greater range of highway traffic operation.

State laws are gradually crowding all advertising, even including that on private property, away from cross roads, railroad grade crossings, and highway curves. Massachusetts now requires roadside fruit and vegetable venders to set up a "Pure Food" sign, but it must not be closer than 500 ft. from the highway. Such laws are, of course, in effect zoning measures, and are extremely helpful. It would seem that states with well organized motor patrols might use them for the control of advertising within the state.

WARNING SIGNS MAY NOT PROMOTE SAFETY

It must also be remembered that there can be such a thing as too many official signs of warning. If there are too many, they will be ignored. Certainly a diminution in the number of trick "Stop" and "Go" signs in many country towns would be desirable.

To obviate the menace of too many gasoline signs, particularly at cross roads and corners, Connecticut licenses roadside filling stations. To further increase safety, motor organizations should be asked to remove their 15-miles-per-hour signs from points where it is unsafe to travel at less than 25 or 30 miles an hour. Certainly the advertiser who continues to appropriate the words "Stop" and "Warning," or who simulates a United States number marker, must be heavily penalized.

There is now under way a distinct movement for wider rights-of-way. The reason for this is twofold—to increase the factor of safety in highway operation and to enhance scenic effects. It is proposed to secure rights-of-way of 400 ft. through public lands on National Park approach roads. The National Forester has recently issued definite instructions to the Regional Foresters in the Western states looking toward the establishment of scenic reserves along forest highways. California is rapidly moving toward the acquisition of a minimum 100-ft. right-of-way plus setback lines in certain areas.

With respect to the improvement of the right-of-way and the forecast of better operating conditions, we are now in a period of transition. Highway operation is not yet stabilized and speed laws are constantly being revised. It is only within the past five years that the practice of snow removal has been adequately developed in many states, and road oil is still carelessly applied. We are now studying the effect of painting tree trunks and telegraph poles white.

Other elements of a parasitic nature, which T. H. MacDonald, Chief of the U.S. Bureau of Public Roads, has characterized as "trespassers," infringe on traffic rights or harass the sensibility of drivers. Among such factors are the unserviceable car, the double rear wheel of the motor bus which runs on the shoulder and kicks dust into the faces of motorists in the rear, and toll bridges. However, the annoyance caused by private toll bridges will probably not continue, as justification for them was lost by the recent introduction of the plan of revenue bonds and public toll bridge authority.

Unprotected Railroad Crossing
Billboards Distract Attention from Warning Sign

My purpose has been to show the sacredness of the highway right-of-way for safe, comfortable, and pleasant operation. I believe that highway operation will presently be subjected to intense study, and that improvements will result. We must look for the invariable characteristics of highway operation in the past, and endeavor to learn what improvement can be made in the future. We will find on careful study certain invariants in the accident story.

In California, for example, for the month of January 1931, the figures for accidents run about parallel to those for January 1930. In other words, it can be predicted that in January 1932 about 2,900 accidents will occur in California, causing about 200 deaths, inflicting injuries on about 3,850, and involving about 4,500 drivers and 1,000 pedestrians. Throughout the United States we find a similar traffic record. If this record can be improved by stricter dealing with highway parasites, it will be well worth the effort.

Committee on Street Thoroughfares Reports

To the Executive Committee
City Planning Division

Your Committee on Street Thoroughfares Manual submits the following progress report.

This Committee was organized early in March 1931. It was first necessary to discuss at some length (1), the scope of the committee's work; and (2) a program for its activities which might promise useful results within a reasonable time as well as establish those results as stepping stones to, or foundations for, a greater and more significant structure.

As to (1)—scope—it obviously will be advisable to avoid overlapping the work of other committees, such as the Subdivision Committee, and therefore it is concluded that, for the present at least, we shall confine our attention to thoroughfares, which shall of course *per se* provide roadways for vehicular traffic, and not canals or railways. And a thoroughfare shall be considered in this connection as a street or road forming a part of a traffic network of a community but not a detail of a subdivision layout. Later, if desirable, this definition may be broadened.

HIGHWAY ALONG THE OREGON COAST

It early becomes evident that for clarity and conciseness some definitions of the terms that will be frequently used or discussed in this connection will be needed. Hence, under (2)—program—one of the first activities will be to consider what may be understood by such expressions as "arterial highways," "boulevards," "parkways," "major (and secondary) highways," "heavy traffic streets," "express streets," "mercantile streets," "major residential streets," "by-pass routes," "plaisances," etc., etc.

It would seem practicable to define some of these and similar terms so that when used they would respectively connote the same idea and not, as now, require long explanations in almost every instance.

In the work of arriving at such definitions as would be generally accepted, it is likely that the characteristics in many cases will be found to be determined by, or to rest upon, certain principles. Of course some terms may be found to be merely of general use—more or less loose—and established from long-convenient, and not unreasonable, custom. The use of these may need to be limited in order to avoid confusion between different localities.

If other terms, founded on certain principles or hypotheses, can in this way have those foundations exposed to scrutiny, we may perhaps not only learn to recognize readily the term itself, and to use it properly, but also to understand better the principles underlying a structure perhaps greater than the term itself.

In presenting the following definitions for consideration, the Committee does so for the purpose of offering concrete expressions for constructive criticism. Such criticism will be most helpful if it shall be in the form of equally concrete suggestions for addition to or modification of the expressions given.

Merely to deny the accuracy or sufficiency of the presented expression will of course indicate the fact of its unacceptableness; but its improvement must most quickly come from a specific suggestion as to the words to be deleted from or added to the given ones.

Written suggestions will be gladly received by the Committee and given the utmost consideration. It should be distinctly understood that the Committee has not yet agreed on these definitions as in their final form but merely presents them now as tentative and for purely temporary use.

DEFINITIONS

ALLEY, a street less than 30 ft. in width, usually in the rear of building lots, and designed for only local service

BOULEVARD, an important thoroughfare of extraordinary width (not less than 100 ft.) with reservations for shade trees and ornamental plantations, and not intended for commercial trucking but readily available for pleasure traffic

BY-PASS ROUTE, a route providing for the passage of traffic around instead of through a locality

*HIGHWAY, the entire right-of-way devoted to public travel including the sidewalks and other public spaces if such exist

ARTERIAL HIGHWAY, a highway that is a main channel, with many tributaries for vehicular traffic, and whose roadway is not less than 40 ft. in width

MAJOR HIGHWAY, a highway forming an essential part of a highway system for a region, such as a state, and whose right-of-way is not less than 100 ft. in width

SECONDARY HIGHWAY, one tributary to or paralleling a major highway and of less importance in the whole system, and/or whose right-of-way is less than 100 ft. in width

PARKWAY, a thoroughfare with not less than 300 ft. width of right-of-way, planted with trees, designed for recreation as well as for pleasure traffic to the exclusion of commercial traffic and trucking, and where egress or ingress to the adjacent private properties is precluded except by supplementary roadways connecting with that of the parkway at only infrequent or widely separated intervals

PLAISANCE, a short parkway whose ornamentation by reservations and plantations is extraordinary, and where parking-spaces for the enjoyment of the views are provided for pleasure vehicles

*ROAD, a highway outside of an urban district

*ROADWAY, that portion of the highway particularly devoted to the use of vehicles

*STREET, a highway in an urban district

EXPRESS STREET, a street devoted to fast traffic

HEAVY TRAFFIC STREET, a street carrying more than the average amount of traffic per square yard of roadway

MERCANTILE STREET, a street devoted to local commercial use

MAJOR RESIDENTIAL STREET, a street through a purely residential section whose roadway is not less than 36 ft. in width

* From the Final Report of the Society's Committee on Road Materials, TRANSACTIONS, vol. 82 (1918), p. 1,384.

The main and general program for the Committee, it is agreed, should be—to quote from the remarks of Hugh E. Young before this Division at Cleveland, July 10, 1930—to produce not "a detailed treatise on how to do 'street thoroughfares' but rather... a statement of principles and perspectives so that those who use it could get the proper relationship between esthetics, usefulness, and foresight, as applied to the various problems of engineering... and economics involved...."

By proceeding along the lines above suggested, your Committee believes the desired ends may be reached advantageously by it. But it also believes that this or any other committee cannot progress satisfactorily, nor will its conclusions be acceptable, unless constructive criticism of its program and work is generously offered contemporaneously by the members of this Division or of the Society at large and by others interested.

Your Committee recognizes gratefully the exceptional opportunity that is offered by this Meeting for presenting its problems at their beginning and for inviting the cooperation of the Society. Frankly, it hopes that this brief report may serve to initiate suggestions or criticisms that the Committee may profit by materially in its work hereafter.

For the Committee on Street Thoroughfares Manual
W. W. CROSBY, *Chairman*

Committee:
Jacob L. Crane, Jr.
Jay Downer
W. W. Horner
Daniel L. Turner

HINTS THAT HELP

Today's Expedient—Tomorrow's Rule

The minutiae of everyday experience comprise a store of knowledge upon which we depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from young and old alike, should afford general pleasure not unmixed with profit.

Latitude and Departure Chart for Azimuth and Distance

By Edward N. Whitney

Associate Member American Society of Civil Engineers
Civil Engineer, Madison, Wis.

SURVEYING texts illustrate 90-deg. coordinate protractors to obtain latitudes and departures from bearings and distances, but by means of the chart here shown one more step may be eliminated, with its accompanying chance of error. On a dam-site and flowage-area survey in 1928, along 20 miles of the Wolf River in Wisconsin, the magnetic needle was used for control, and the transit was usually set up at every other station. At the start of the work I prepared a chart, illustrated in Fig. 1, for obtaining the latitudes and departures of courses directly from the course distances and azimuths, which in this survey were referred to the south. This graphic method eliminates the work of changing the azimuth to a bearing, looking up the sine and cosine of the bearing angle, and making the two multiplications by the course distance.

The original chart, about 60 in. long and 42 in. high, was made on cross section tracing cloth with ten divisions to the inch. The scale used was 1 in. = 10 ft. For distances over 500 ft., the scale was changed as required. Blue prints for the charts used in the field office were mounted on wall board. Azimuth lines at intervals of 15 min., 30 min., or 1 deg. were drawn, so that there would not be an overcrowding of converging lines. Since the chart covers 45 deg. of arc, 8 settings of the azimuth strip are provided to cover the full 360 deg. required.

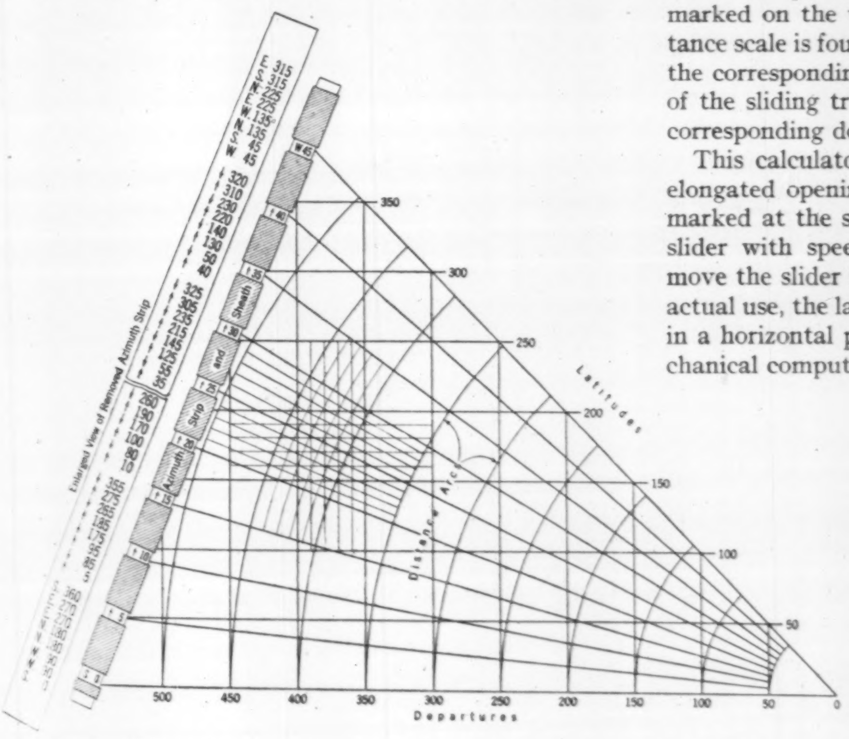
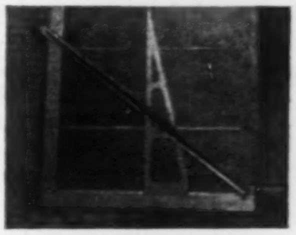


Fig. 1. Chart for Obtaining Latitudes and Departures from Azimuths

The strip with the azimuth index is mounted so as to slide up and down in a sheath of heavy detail paper, which is fastened permanently to the chart. This sheath has openings opposite the 5-deg. divisions of the chart, so that for any one setting of the slider the azimuth numbers appear in proper sequence from bottom to top, or from top to bottom. Arrows near the azimuth numbers indicate the direction of the numbering. The correct letter—N., S., E., or W.—also appears at the bottom and top openings of the sheath. Large darning needles help to locate the required point on the chart while the latitudes and departures are read and recorded.



A Mechanical Computer
Developed by
Prof. Ray S. Owen

The photograph shows this idea as it was worked out mechanically by Prof. Ray S. Owen, of the University of Wisconsin. The square base is of wood, one inch thick. All other parts are metal. Azimuth graduations are marked on the vertical strip at the left side. The distance scale is found on the pivoted hypotenuse arm, while the corresponding latitude is read from the vertical side of the sliding triangle that acts as a T-square, and the corresponding departure along the bottom.

This calculator has an additional feature, a separate elongated opening with the eight positions of the slider marked at the side. It permits the operator to set the slider with speed and accuracy. The handle used to move the slider is seen at the left side of the frame. In actual use, the latitude and departure calculator is placed in a horizontal position on a table. The chart and mechanical computer can also be used in the office or classroom to check trigonometric computations, particularly of the right triangle.

Both chart and calculator have been used by Professor Owen in the university class room, and in summer schools of surveying. He has found both types useful and satisfactory. Evidently, with modern means of transportation, the civil engineer can now take with him to field offices, devices and conveniences that formerly were left behind with his grand piano and other marks of civilization.

Simplifying Structural Steel Design

By VSEVOLOD KHMELEVSKY
DESIGNER, THE NEW YORK CENTRAL RAILROAD COMPANY
NEW YORK, N.Y.

ONE of the designer's most effective mechanical calculators is the slide rule. Besides the very common logarithmic form, there are various types of slide rule which are adapted for use in many branches of engineering, such as hydraulics, chemistry, electro-technics, surveying, and concrete design.

A great deal of structural steelwork, such as bridges and buildings, consists of girders and beams of compound or rolled section. The designing of such girders may be divided into two parts: calculation of the stresses due to imposed loads, and determination of the section. For complete design, it is also necessary to calculate the web and other splices, as well as the length of cover plates, stiffeners, and billets, but the practice has many simple methods to solve these problems.

The first part of the design depends largely upon the type and system of loading. Because of the great number of non-standard loadings, each particular case must be considered separately. Certain mathematical expressions are used to study the various conditions under which structural shapes are subjected to stresses. From the result of such computations the value of the necessary cross section, in accordance with the allowable unit stress, may be determined.

PLATE GIRDER SLIDE RULE

A slide rule to perform the second part of the design, which can give the cross section for any girder, has been developed by the author. It does not depend upon any system of loading because it is based on the moment of inertia method. The girder can have any possible depth and can be with or without cover plates, but must have a symmetrical section of four angles. For other combinations, it is only necessary to have a complete set of scales for different sizes of angles. But practically speaking, this complete set consists only of two sizes of angles, 6 by 6 in. and 8 by 8 in., with different thicknesses, because other angles for girder sections are used very seldom. The scale for other angles can be located on the rule in the same way.

In the accompanying illustration, Fig. 1, both sides of this slide rule are shown. It is capable of solving a section of girder having 6 by 6-in. angles, varying in thickness in $^1/_8$-in. steps from $^3/_8$ in. to 1 in., with girder depths of from 24 to 120 in., back to back of angles, and a thickness of cover plates from 0 to $2^1/_4$ in. The scale of the section modulus covers from 150 to 6,200 in.^{3} on the middle movable part, where are also shown the indexes (with a circle) for different thicknesses of angles. On the upper part there are scales of depth of girder and of thickness of cover plates (in eighths of inches) for the net section, without the holes for rivets (in tension), and on the lower part are corresponding scales for the gross section (in compression). On the right side of the rule there are given the scales of net (assuming 3-in. pitch for web rivets) and gross section modulus of web for different depths, to eliminate the influence of the thickness of the web, which must be determined usually by the maximum shear in the girder. For more accurate readings the vertical line on the runner may be used.

In use, the scale of the middle part is moved until the index on the left side matches the given section modulus. By sliding the line on the runner up to the index of the desired angle, the necessary depth of girder and thickness of cover plates for an average of 36 possible combinations of angles and plates for different depths may be read.

The reverse problem is to find the section modulus for a given section. It is solved by matching the corresponding index with the given section on the scale, which gives the definite reading of the section modulus on the movable part opposite the index on the left side. The other indexes give the readings for all possible combinations of a section equivalent to the given one, without any additional movement.

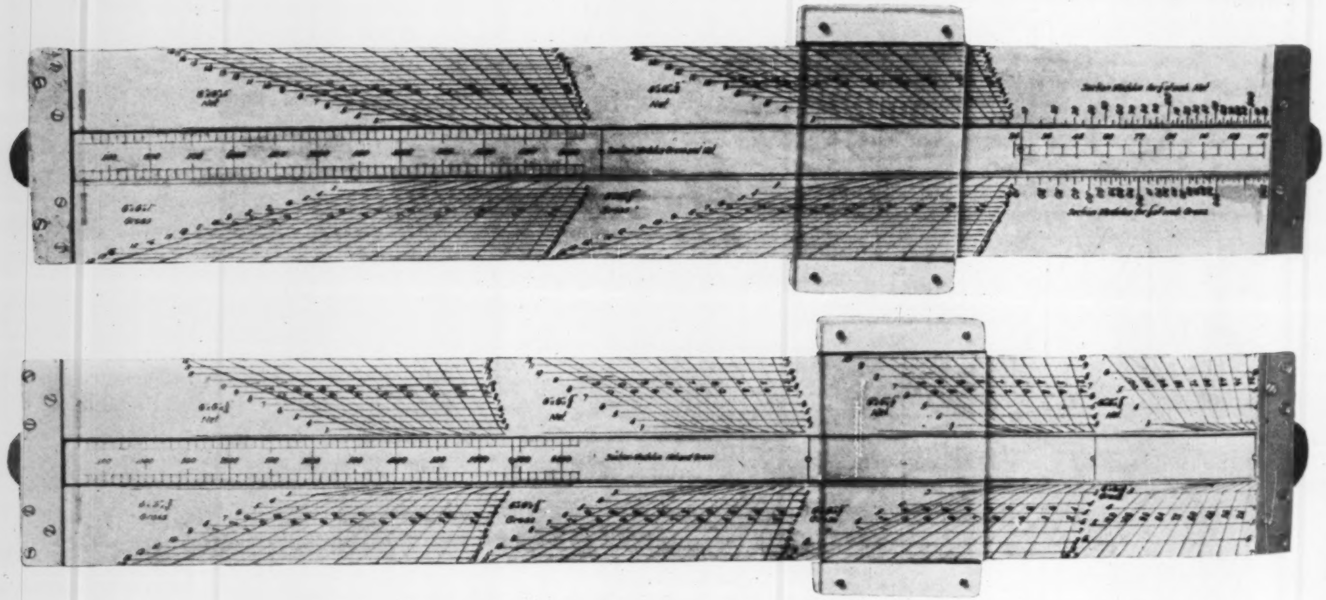


FIG. 1. PLATE GIRDER SLIDE RULE FOR A GENERAL CASE (BOTH SIDES)

Besides solving these two main problems, the rule makes it possible to examine all limits of section modulus for different combinations of depth, angles, and cover plates. By the regular method of computation, this study would require a great amount of time. In addition to time saving, this slide rule has another advantage. The possibility of mistakes in collecting data from different tables, gathering and adding them, is eliminated, since only one setting of the rule is necessary.

SLIDE RULE FOR DECK PLATE GIRDER BRIDGE

In Fig. 2 is shown a slide rule which was developed for use in designing deck plate girder bridges with standard loading (Cooper's E-70) for spans from 20 to 100 ft. It gives all the designing data, such as the maximum bending moment, the maximum shear, the section modulus required and obtained, the cross section of each girder, and the total weight of the bridge. A girder can have any possible depth and can be with or without cover plates, but must have a symmetrical section of four angles. It is not for a general case as is the first slide rule, but it entirely eliminates any computations. By one setting for the length of a span the readings of the scales give all the elements of the design.

On the middle movable part is located the scale of span length. On the upper and lower parts of one side are shown the scales of maximum shear, maximum moment, section modulus required, and total weight of bridge. On the other side are the section modulus obtained and the cross sections of girders for three relations of depth to span—1:10, 1:12, and 1:15. The last scale is made in accordance with these ratios and gives the economical section for girders in three variations and for the two most usual flange angles, 8 by 8 in. and 6 by 6 in., with a limited thickness of cover plates.

This slide rule was based on a large number of actual designs of bridges already built. As an example, in checking the design of a 62-ft. deck plate girder bridge for one track, having a depth of one-tenth of the span, the designer must ordinarily calculate the whole design, using tables or the analytical method, and calculating all the elements. For these calculations, an experienced man would require three or four hours. By means of the slide rule any man who had used it once would require not more than three or four minutes.

Other uses of these two types of slide rule will suggest themselves to the bridge engineer. These handmade slide rules are about 12 in. long.

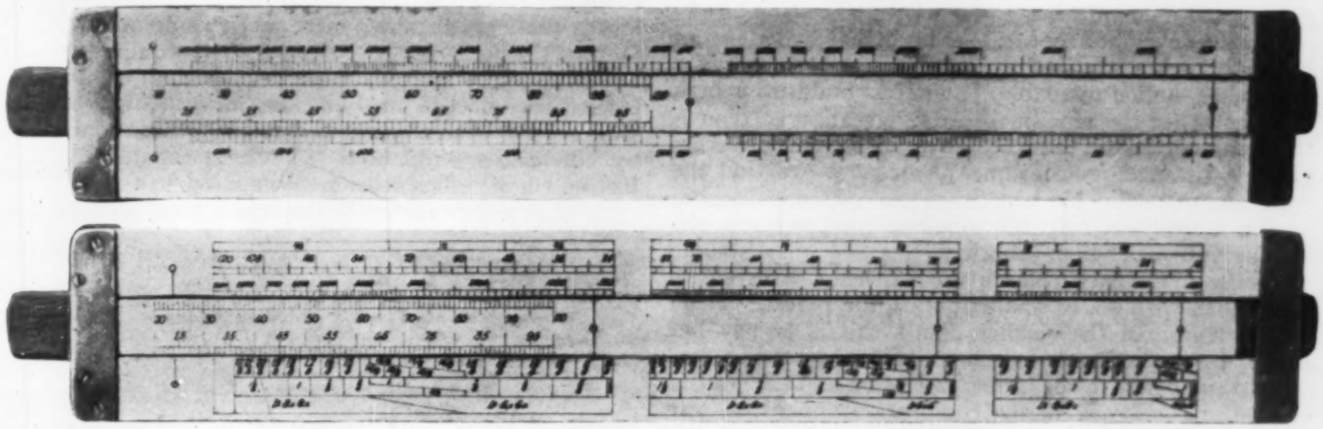


Fig. 2. Deck Plate Girder Slide Rule for a Particular Case (Both Sides)

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

Importance of Intracoastal Waterway

Editor: The interesting paper by Mr. Young, "The Atlantic Intracoastal Waterway," in the June issue, indicates that a great deal has been accomplished on the project and also that work on the remaining sections of the waterway is being pushed to completion. I hope that the Atlantic and Gulf intracoastal waterways will soon be connected as an integral whole from Boston to Corpus Christi.

The new sections now being finished from Beaufort, N.C., to Winyah Bay, S.C., will greatly increase the traveling facilities of small craft and pleasure boats. On several occasions, I have encountered storms outside inhospitable sections of the Atlantic Coast in not too seaworthy boats and have wondered whether I could reach land or not. Such incidents make one a strong booster for inland waterways.

The present-day strides in the development of small and medium-sized Diesel-engined tow boats for handling tows in shallow water, and of self-propelling shallow draft Diesel-engined cargo boats make the inland waterway more able to compete with railroad transportation than heretofore, as is indicated in Mr. Young's paper.

The improvement of the waterway between Jacksonville and Miami is badly needed. I had occasion to use part of this channel a few years ago, and the traffic tie-ups resulting from narrow channel width and shallow water made a trip through the channel one of numerous delays and hardships.

An inland waterway across the neck of Florida has been talked of for a great many years. There appear to be no serious difficulties in the way of such a channel. The local traffic would be considerable, as is indicated by the fact that the Jacksonville-Palatka and Palatka-Lake Harney sections recently deepened show, according to the report of the chief engineers for 1928–29, a 90 and 91 per cent increase. Intracoastal through traffic to the Southern Gulf ports would probably increase the traffic through the whole length of the waterway. Certainly there would be an increase in tourist or pleasure-boat traffic.

O. P. ERICKSON, M. Am. Soc. C.E.
Great Lakes Dredge and Dock Company

Chicago, Ill.
July 17, 1931

Importance of Hydrogen Loss in Boilers

EDITOR: The gas industry is to be congratulated on Mr. Rhodes' careful evaluation of natural gas among power fuels, published in the March issue of CIVIL ENGINEERING. Although Mr. Rhodes plainly seeks to exploit the advantages of natural gas, he has generously recognized its limitations inherent in long transport systems and those resulting from the difference between supply and demand.

A point developed by Mr. Rhodes is that by having an auxiliary fuel, a large industry can absorb the dump gas "when, as, and if available," and thus build up a high load factor for the pipe line. His suggestion is tremendously important in that it reveals a method of delivering gas to a distant large consumer at a cost so low that the consumer can enjoy the benefits of gas economically and, at the same time, help the pipe-line company pay the investment sooner than otherwise.

While in the large plants the efficiency with gas is lower than with oil, in smaller plants the reverse generally is true. In the smaller plants this is largely because the gas companies render real engineering service to customers in the proper operation and care of the equipment, not only in the combustion apparatus, but in the application of heat and power generally, while the fuel oil companies usually sell nothing but fuel.

I do not agree entirely with Mr. Rhodes' contention that oil firing must be done with greater excess air than gas. In a major plant with which I am familiar, the boilers, both with water walls and refractory walls, are fired regularly at ratings of from 100 to 500 per cent with approximately 10 per cent excess air with either fuel.

It is my opinion that an efficiency of 85 per cent, or even of 83.5 per cent, with an air preheater, is more ideal than actual. Even with an air preheater and 300 deg. flue temperature, such an efficiency would demand total radiation and undetermined losses of less than 2 per cent. Furthermore, 300 deg. is about the minimum practical temperature, and for ordinary overload at operating points, the exit temperatures will be nearer 400 or 450 deg. I doubt that Mr. Rhodes is serious when he says that the hydrogen loss is unimportant. The importance of this loss can be more clearly emphasized by showing the total hydrogen loss, instead of only the latent heat loss, in the following formula:

$$\text{B.t.u. per lb.} = \frac{9 \times \text{per cent hydrogen}}{100} \times [(212\text{ deg.} - Ta) + 970.4 + 0.48\,(Tg - 212\text{ deg.})]$$

in which Ta = Temperature of fuel and air
Tg = Temperature of flue gas

Or, setting these values out in table form:

TABLE I. FUEL VALUES (RHODES)

	NATURAL GAS	FUEL OIL	ALABAMA COAL
Gross B.t.u. per lb.	21,000	18,800	13,200
Per cent hydrogen	21.5	12.5	4.8

TABLE II. HYDROGEN LOSSES (RHODES)

	NATURAL GAS		FUEL OIL		ALABAMA COAL	
	B.t.u. per lb.	Per cent	B.t.u. per lb.	Per cent	B.t.u. per lb.	Per cent
Latent heat loss	1,932	9.2	1,128	6.0	436	3.3

TABLE III. TOTAL HYDROGEN LOSSES (O'NEILL)

	NATURAL GAS (HYDROGEN LOSS)		FUEL OIL (HYDROGEN LOSS)		ALABAMA COAL (HYDROGEN LOSS)	
FLUE GAS TEMP. DEG. FAHR.	B.t.u. per lb.	Per cent	B.t.u. per lb.	Per cent	B.t.u. per lb.	Per cent
300	2,220	10.6	1,300	6.9	499	3.8
400	2,320	11.1	1,327	7.1	520	3.9
500	2,415	11.5	1,410	7.5	540	4.1
600	2,510	11.9	1,460	7.8	560	4.2
700	2,610	12.4	1,520	8.1	582	4.4

At a 500-deg. flue temperature, this shows the hydrogen loss detriments of natural gas to be 4 per cent and 7.4 per cent compared to 3.2 per cent and 5.5 per cent, as indicated by Mr. Rhodes, for fuel oil and coal, respectively. One great reason for emphasizing hydrogen loss is that, even today, there are gas-appliance manufacturers who covertly compare their equipment efficiencies on gas charged with low heating values with efficiencies on coal and oil charged with high heating values. For several years we have advocated evaluating the efficiencies on the basis of net heat value for all fuels just as engine efficiencies are compared to the Rankine cycle of available energy in the steam.

HAYLETT O'NEILL

Houston, Tex.
July 22, 1931

Novel Surveying Methods

EDITOR: The article by Colonel Birdseye, "Photographic Surveys of Hoover Dam Site," in the April issue of CIVIL ENGINEERING, draws attention to what may well prove to be a useful tool to engineers. Although photographic methods—particularly aerial—have ceased to be novel, the work at Hoover Dam is believed to be the first instance of the use in the United States of the modern photo-theodolite for large-scale mapping.

Prior to final authorization of the Boulder Canyon Project, the most efficient use of the funds appropriated for investigation, and the general status of the project, limited the amount of money to be spent on topographic surveys. As a result, the topography available at the time construction was finally authorized was preliminary in nature; and while well adapted to the preparation of designs, it lacked the accuracy necessary for precise estimates of final quantities under a contract, and covered less area than was essential for a project.

The great height of the Hoover Dam, the steepness of the canyon walls, which, in places, average about $^1/_4$:1, seldom flatter than $^1/_2$:1, for 600 to 700 ft. above the river bed, and the narrowness of the canyon, combined with the ruggedness of the surface, made the successful use of ordinary topographic methods doubtful. The limited time available for issuing specifications, after the project was authorized and funds secured, made their use out of the question. As it is impossible to obtain satisfactory photographs from an airplane, the only solution seemed to be the use of terrestrial photography, and the work was accordingly done by this method.

Check profiles taken over some of the more accessible parts of the area covered showed very fair agreement with the topography. The maximum discrepancies seemed to lie, as might be expected, on the flatter slopes, for which the position determination, based on parallax, appeared to be a trifle less accurate than was desirable. This was doubtless due principally to the very limited length of base available.

It is believed that, with the improvements in technic which will probably follow an increasing use, the phototheodolite will prove superior, for difficult locations, to the older tools of the topographer in speed, accuracy, and cost.

D. C. McConaughy
Senior Engineer, U.S. Department of the Interior, Bureau of Reclamation

Denver, Colo.
August 5, 1931

Duchemin's Formula and Resultant Pressures on Structures

Dear Sir: The preliminary report of the Structural Division's Subcommittee on Wind Bracing in Steel Buildings not only contains matter of merit but also is causing considerable discussion which will appreciably enhance its value.

There is need of comprehensive experimental wind pressure data applicable to buildings and roofs of different types and proportions, and also to skeleton steel structures of various kinds. For inclined roofs, on the surfaces of which the pressure not only varies in intensity but also changes sign, no formula appears to be available at present which will readily and fully express the pressure intensities and fluctuations involved. In the absence of a simple expression fully applicable to inclined roofs and other types of structures, the use of the Duchemin formula, as supplementary to available experimental data, would appear to be warranted and helpful in making use of such experimental data, keeping in mind the conditions for which the formula was derived.

A further need is for fuller knowledge of the velocity and pressure characteristics of gusts and different kinds of fluctuating winds. Such knowledge would assist in making adequate provision in safeguarding against that wind which gives the critical load, without making excess provision for it. It is evident that both the intensity and duration of gusts and also the characteristics of the structure acted on are factors in determining the critical wind in an individual case.

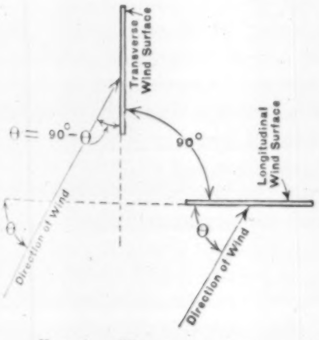


Fig. 1. Hypothetical Structure

Duchemin's formula gives pressures for inclined wind on flat plates considerably in excess of those corresponding to the resolution of forces. Experimenters generally recognize that a portion at least of the high total pressure on inclined plates is due to the vacuum at the rear. As the maximum vacuum cannot exceed an amount equal to atmospheric pressure, the ratio of vacuum to total pressure in high winds may be much less than for low winds.

The ratio of vacuum which occurs with wind on flat plates to that on other flat surfaces, such as those of buildings and roofs, would evidently account in part for the difference in total unit pressures on the surfaces involved.

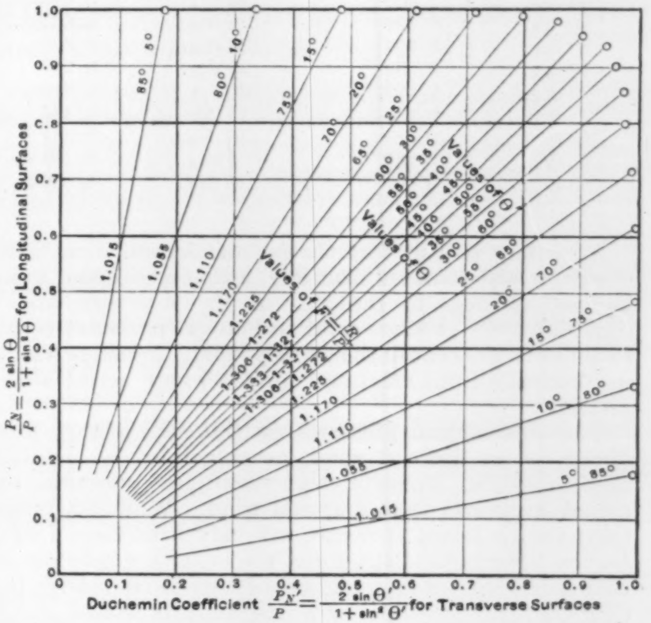


Fig. 2. Resultant Pressure Factors for Diagonal Wind

Duchemin's formula for normal pressure on thin flat plates, stationary and inclined to the wind, is

$$P_N = P \frac{2 \sin \theta}{1 + \sin^2 \theta}$$

in which P_N = Pressure normal to the surface per unit area of surface pressed.

θ = Angle between axis of wind and surface pressed.

P = Pressure per unit area on a surface at right angles to the wind.

Table I gives the values of $\frac{P_N}{P}$ for various values of the angle θ.

The resultant pressure factors for a diagonal wind acting on a hypothetical structure, Fig. 1, assumed to have equal longitudinal and transverse effective wind surfaces, such that Duchemin's formula for inclined wind would be applicable, are plotted in Fig. 2. In deriving the resultant factor,

$$F = \frac{R}{P} = \left[\left(\frac{P_N}{P}\right)^2 + \left(\frac{P_{N^1}}{P}\right)^2\right]^{\frac{1}{2}} = \left[\left(\frac{2 \sin \theta}{1 + \sin {}^2\theta}\right)^2 + \left(\frac{2 \sin \theta^1}{1 + \sin {}^2\theta^1}\right)^2\right]^{\frac{1}{2}},$$

in which $R = [(P_N)^2 + (P_{N^1})^2]^{\frac{1}{2}}$, it has been assumed that the direction of the wind is parallel to a plane which is normal to the longitudinal and transverse wind surfaces and, to meet the conditions under which Duchemin's formula was developed, that the wind surfaces are the equivalent of thin flat plates.

TABLE I. VALUES OF DUCHEMIN'S COEFFICIENT

$$\frac{P_N}{P} = \frac{2 \sin \theta}{1 + \sin^2 \theta}$$

θ	$\frac{P_N}{P}$	θ	$\frac{P_N}{P}$
90 deg.	1.000	40 deg.	0.910
85 deg.	0.999	35 deg.	0.863
80 deg.	0.999	30 deg.	0.800
75 deg.	0.999	25 deg.	0.717
70 deg.	0.998	20 deg.	0.612
65 deg.	0.995	15 deg.	0.485
60 deg.	0.989	10 deg.	0.337
55 deg.	0.980	5 deg.	0.173
50 deg.	0.965	0 deg.	0.000
45 deg.	0.943		

According to this formula for the hypothetical case shown, it will be noted from Fig. 2 that a diagonal wind inclined 45 deg. to the axis of the structure would simultaneously produce axial and lateral pressures equal to about 94 per cent of the corresponding pressures which would result from axial and lateral winds acting separately.

The high normal pressures on thin, flat plates, produced by an inclined wind, as calculated by Duchemin's formula, indicate high resultant pressures on actual structures from diagonal wind. While the resultants for an actual structure will vary from those found for the hypothetical structure, the latter are believed to sufficiently approach actual conditions to emphasize the fact that the possible high resultant pressures merit careful consideration.

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Valley Railway Company

Pittsburgh, Pa.
August 1, 1931

Triangulation and Modern Progress

EDITOR: The excellent paper in the June issue by Captain Patton, Director of the U.S. Coast and Geodetic Survey, entitled "The Utility of Geodetic Control Surveys," should be read carefully by every engineer engaged in large projects involving the use of land, for it contains much that is useful.

When an engineer is planning the location and construction of some great project, he should know where his land is. He should know its boundaries and whether or not those boundaries have been properly defined in terms of some system of coordinates. If they have not been so defined, it may be a very expensive matter to pay damages for encroachment to the owner of abutting property. The days of slipshod surveying and mapping have been relegated to the past by the increased value of land and the enormously increased expenditures for engineering construction. Even in remote districts land may be worth thousands of dollars an acre, as is the case in oil fields. The rights-of-way of highways and electric power, telephone, and telegraph lines can best be secured and perpetuated by having an accurate knowledge of the geographic position of the terrain. All of this can be accomplished when alignment surveys and surveys of the boundaries of private or public property are tied into the great triangulation net of the country, which is rapidly nearing completion.

This does not mean that the engineer must carry on extensive triangulation operations every time that he makes a property survey. However, he should tie his property boundaries into a triangulation or a traverse station that has already been coordinated with the triangulation net or, in the absence of any fundamental horizontal control stations in the region, he should put in substantial monuments on or near the boundary which can later be tied into the national horizontal control net. Wider use of the triangulation net of the country and of traverses based on that net would be a step toward the elimination of waste in many lines of human activity.

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Washington, D.C.
July 17, 1931

Aggregate Is Important

TO THE EDITOR: In his excellent paper, "Basic Principles of Concrete Making," in the April issue, Mr. McMillan has given very clear evidence that the compressive strength, water-tightness, flexual strength, and durability of concrete are dependent on the properties of the cement paste. However, he emphasizes the fact that no matter how skillfully the concrete mixtures are designed, these potential qualities will be lost unless the concrete is properly handled and placed. Concrete mixtures very carefully designed in accordance with modern methods, but carelessly placed and cured, would probably result in a less durable structure than if an inferior mixture, properly handled, were used.

It seems most unlikely that the condition of concrete in a standard specimen at the date of test is truly representative of the same concrete in the structure, as the quantity of coarse aggregate, the methods of compacting, and the curing conditions may all be different. Further, the fact that laboratory specimens show strengths equal to, or above, specification requirements may lead to an unwarranted sense of security which may easily tolerate careless handling of the concrete in the field. Why not direct our efforts in the laboratory toward a careful control of the qualities of cement paste, or mortar, and give more attention to proper placing?

During the era when the maximum size of coarse aggregate seldom exceeded $1^1/_2$ or 2 in., it was possible to discuss and compare the various qualities of different concretes on common ground. However, since the tendency in recent years seems to be toward the use of coarse aggregate of a much larger size, difficulties in testing have necessitated the alteration of mixtures by the removal of certain sizes of coarse aggregate particles, which undoubtedly affects some of the properties of the concrete. For example, in a gravity dam the concrete is to contain coarse aggregate as large as 6 in., and the compressive strength has been specified at 3,000 lb. per sq. in. at 28 days. Abram's curves indicate that this strength should be obtained by using a water–cement ratio equal to about $6^1/_2$ gal. per bag of cement. In order to follow standard practice in regard to the size of test specimens, from 60 to 70 per cent of the coarse aggregate is removed before forming the sample into 6 by 12-in. or 8 by 16-in. cylinders. At the age of 28 days the strength may be 3,800 lb. or better. If the water–cement ratio is lowered to where the modified test specimens will give the designed strength, it is very doubtful whether the concrete will be as water-tight as desired. Compressive strength is specified merely on account of the lack of a better yardstick with which to measure other desirable properties, the principal requirements being water-tightness, low shrinkage, and weight. The ideal mixture in this type of structure would be one in which the largest possible quantity of coarse aggregate is bound together by the minimum amount of mortar of very high quality. Such a mixture, regardless of the water–cement ratio and the size of the test specimen, would show compressive strength much lower than that predicted by Abram's curves.

As Mr. McMillan has pointed out, a definite relation exists between the strength of cement paste and the qualities of concrete made from that paste, but the relation is not the same for all classes of mixtures and all types of aggregates. The ratio of mortar strength is greatly influenced by the ratio of the quantity of mortar to the quantity of aggregate. A concrete mixture carrying a heavy excess of mortar of a given water–cement ratio would give higher compressive strength than one in which the voids in the coarse aggregate were exactly filled by a mortar of the same water–cement ratio, although the latter mixture would be more desirable.

While Mr. McMillan's paper invites consideration and study of the properties of cement paste (cement and water), it is suggested that a full understanding of this paste plus a third ingredient, fine aggregate, might be of greater value in interpreting the qualities of concrete made from the mortar combination.

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July 15, 1931

New Formula for Reinforced Concrete Columns Needed

To the Editor: In connection with the article by Messrs. Gilkey and Raeder, in the July issue of Civil Engineering, the following points may be of interest. Since test data on concrete beams and columns have accumulated over a period of 35 years, it is strange that no particular effort has been made to remedy the serious disparity known to exist between steel and concrete stresses assumed for design and those which develop after plastic flow and shrinkage have taken place.

According to recent tests on columns under the action of sustained loads, the vertical steel was stressed to the elastic limit, while the initial concrete stress was lowered, due to plastic flow and shrinkage in the concrete. Consequently the only effective structural element in such columns is represented by the stays or hooping, and whether these are sufficient to prevent buckling of the vertical steel and consequent bursting of the column will depend on the amount and disposition of the hooping material. In the case of stayed columns this is never true, while spiral hooping may answer if it is properly proportioned.

Reinforced concrete beams and slabs sag with age, and columns will shorten—especially if they are continuously dry—so that, in this respect, no new difficulties will be encountered. It is apparent that concrete columns, designed without any vertical steel, would be quite immune from the deleterious effects of flow and shrinkage, provided the small change in column length is not objectionable.

My solution to the column problem would tend to render the design safe and permanent, minimize flow and shrinkage, and eliminate the modular ratio n, as a factor in design. This solution consists in designing a concrete column with spiral reinforcement sufficient to resist the entire bursting force on the basis of, say, 18,000 lb. allowable unit tensile stress, and providing only enough vertical stays to support the hooping. This is exactly the reverse of the prevalent method of design. The process would thus be based on the ultimate unit compressive strength of the concrete f_c, which will govern the required concrete area A_c, for a given column load P, making $A_c = \frac{P}{f_c}$. Then, the required hooping area a_s, as a function of f_c, may be determined. This function is as yet unknown, but can be ascertained from a series of appropriate tests. The hooping steel for this purpose should consist of flat bars instead of the customary rounds, using a maximum pitch of spiral just sufficient to resist the flow of concrete through the open spaces and yet sufficient to make a good bond between the column core and its insulation.

If we were dealing with a frictionless fluid, such as water, instead of concrete, this principle of design would be quite simple. For example, a steel tube of radius r, thickness t, filled with water supporting a load P, on water-tight pistons top and bottom, neglecting friction, could be figured in the following way. The unit pressure on the cylinder content would be $\frac{P}{\pi r^2}$, producing a hoop tension $\frac{P}{\pi r}$, and requiring a thickness of metal $t = \frac{P}{f\pi r}$, and a required steel area of $\frac{2\pi r P}{f \pi r} = \frac{2P}{f}$, which is a constant for any load P. Hence, for $P = 1{,}000{,}000$ lb., and $f = 18{,}000$ lb., the steel area would be 112 sq. in., requiring a cylinder of any diameter with a corresponding thickness to give this area. Thus for a diameter of 35 in., the thickness would be 1 in., and the pressure on the water would be 1,000 lb. per sq. in.

If concrete were substituted for the water, most of the lateral pressure would be absorbed by internal friction or stress, and only a small fraction of the unit pressure would remain to exert hoop tension. In the absence of the tension element, the concrete core would burst when the load P attained a certain magnitude. By proper reinforcement to resist bursting, such columns may be made to sustain almost any load and the shortening due to shrinkage and flow can be reduced to a minimum. This analogy shows that a concrete column can be designed for any load, possibly exceeding the crushing strength of the concrete, with a relatively thin enclosing steel cylinder, or spiral hooping to resist the bursting tension, without carrying any vertical load on the steel. Of course, square and rectangular columns would require cylindrical cores and the ordinary stayed column would be a thing of the past.

At present, few tests are available to substantiate these claims, but such tests as have been made on columns without vertical steel, show conclusively that the hooping can be made to resist the entire column load. This also explains the inferiority of stayed columns, since stays are very poor tension members. (Bulletin 300, University of Wisconsin.) The subject here treated should prove a fertile field of investigation for Committee 105 of the Concrete Institute, and should lead to a satisfactory and economical solution of the column problem.

David Molitor, M. Am. Soc. C.E.
Consulting Engineer

Detroit, Mich.
August 10, 1931

De Gaetani's Galleried Dam

SIR: In his recent article on "Honeycomb Gravity-Type Concrete Dams," in the April issue, Mr. Grunsky outlined a type of design which, for very high dams, certainly merits further investigation and consideration. The advantages claimed as to heat control during construction and the reduction, if not the elimination, of uplift are obvious. In view of the absence of metal reinforcement which might corrode, the suggested metal diaphragm in the upstream face, though desirable, does not appear essential to the design.

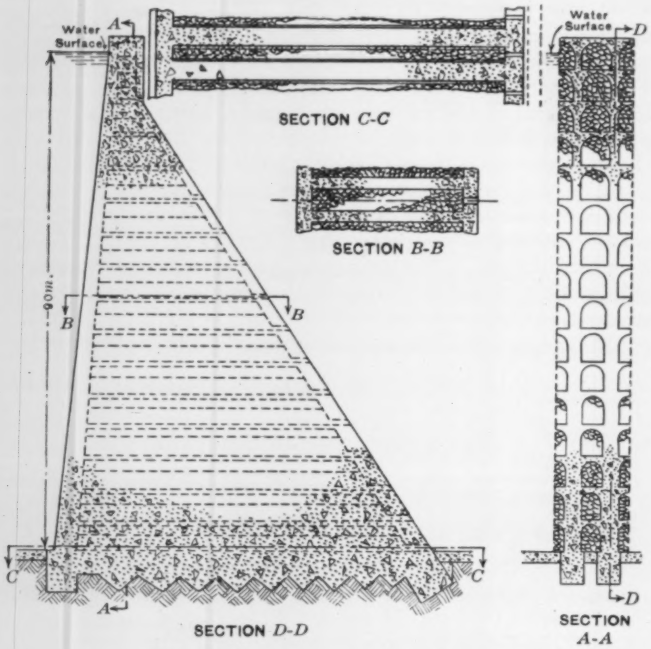

FIG. 1. GALLERIED GRAVITY DAM SUGGESTED BY DE GAETANI IN 1924

As regards detail design, it would no doubt be necessary to consider very carefully the distribution of stresses at the junction of the massive face slab and the lighter honeycomb galleries, with a view to preventing the development of local shear cracks at this point. I would prefer that the partition walls between cells in each horizontal set of galleries be placed vertically over the walls of each lower set, so that these walls would, in effect, form continuous buttresses throughout the structure, with, of course, the proposed horizontal joints between the various sets of galleries. The width of cells would decrease from the top of the dam to the bottom, so that the partition walls forming these buttresses would increase in thickness with the load. A special type of gravity dam embodying some of these principles was suggested some years ago by Edoardo De Gaetani in an article, "Di Uno Speciale Tipe di Diga a Gravita," published in *Annali dei Lavori Pubblici*, November 1924.

In brief, De Gaetani's object was the attainment of a design less costly than a plain gravity dam with, at the same time, elimination of underpressure, and without the introduction of metal reinforcement. As may be seen by reference to Fig. 1 in this letter, his design had approximately the same profile as a plain gravity dam. It comprised an upstream face wall of varying thickness constructed of rich concrete to prevent infiltration, a downstream wall of ordinary concrete with drainage openings, and an intermediate structure of counterforts on independent footings keyed into the rock foundation, with transverse arches tying these counterforts together. A necessary feature of De Gaetani's design was the filling of the compartments with a dense sand, gravel, or rock mixture, which would provide weights for stability while permitting free drainage through the structure. He claimed that the adoption of this type of dam would result in a saving in concrete of more than 40 per cent, and that the internal stresses would be less than those in an ordinary gravity dam of equal size.

In his published description, De Gaetani referred to a dam under construction on the River Orba near Zerbino which, he said, had a certain resemblance to his proposal. I am, however, unable to obtain particulars of this structure.

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Engineer, State Rivers and Water Supply Commission

Victoria, Australia
July 5, 1931

Additional Factor of Safety for Dams

EDITOR: The design of a honeycomb gravity-type concrete dam with inclined upstream face, described by Mr. Grunsky in the April issue, is an attempt to distribute the stresses in a gravity dam more efficiently. The resultant of all the forces apparently cuts the base at or near the middle of the section, thereby at least doubling the ordinary factor of safety for a gravity dam. This is accomplished with approximately the same amount of concrete as is used in the ordinary massive gravity dam plus some rockfill in the galleries. The form cost is probably twice as high for the honeycomb gravity-type as for the ordinary type of gravity dam, but this item would probably be less than 10 per cent of the total cost, therefore the doubling of the safety factor has been obtained with a very small additional cost. It is a foregone conclusion that a larger factor of safety is more needed in a gravity dam than low average stress.

Also, Mr. Grunsky recommends that the dam be built straight in plan with a sloping upstream face. The sloping upstream face does more to increase the factor of safety than does arching. Furthermore, this increase is constant, whereas any probable increase in safety obtained by arching varies from a maximum in late summer to nothing in late winter. Consequently in late winter the structure has to depend upon its gravity action for safety. It has also been shown, both analytically and by tests, that a gravity dam should, for the maximum factor of safety, be arched downstream and not upstream. (*Transactions*, 1928, page 785, and *Proceedings* for March 1931.) Since the straight dam is the simplest structure possible and since a heavier section can be thus built using the same amount of material, no apparent reason seems to exist for arching a gravity dam in plan—especially upstream. I do not feel that it is necessary or desirable to cool a straight dam. The artificial cooling will increase the cracking tendency of the concrete, as the shrinkage is brought about at an earlier period, when the concrete has less tensile strength than later on. Therefore, I feel that Mr. Grunsky's tentative spacing between the contraction joints of 68 ft. is rather large, if it is intended to avoid cracks altogether.

Since the concrete will shrink in an up- and downstream direction also, it will be necessary in large dams to have contraction joints running lengthwise spaced 68 ft. or so apart. These joints should be inclined toward the upstream face, in line with the principal stress, in order to preserve the full shearing strength of the concrete.

It should not be forgotten that the dam sections on the hillside have to take the extra load transferred to them from the midstream portion on account of the difference in deflection under load. I feel that a simple way to do this on a gravity dam is to provide one or several buttresses on the downstream side in order to increase the moment arm at the places where the overturning force is greater than that due to the local water load. Such a provision has been made in several places. For a gravity dam arched upstream, the portion on the hillside toward the haunches is usually located on a downhill slope and needs an addition to the moment arm on that account, either in the shape of buttresses or some addition to the section.

Lars Jorgensen, M. Am. Soc. C.E.
Consulting Hydro-Electric Engineer

San Francisco, Calif.
July 19, 1931

Personal Equation in Angle Measurement

Editor: I have been greatly interested in the articles on surveying in recent issues of Civil Engineering. In his letter, "Astigmatism a Cause of Error in Surveying," in the June issue, Mr. Fee is undoubtedly correct in assuming that greater accuracy is obtained by not setting the plates to zero. It is also quite probable that the personal equation of an observer may be different when setting "zero" than when reading an angle, for the two operations, from an optical standpoint, are distinctly different.

In geodetic work, where a high order of accuracy is desired, the plates are not once set to zero or any exact degree, even at the start of a set or position. Instead, an approximate setting only is made. The exact readings on the initial are then taken and recorded in the same manner as that used on all later pointings. This procedure makes the effect of the personal equation constant and eliminates it from the resulting angles. The same principle obtains in the use of either a repeating or direction type of instrument.

As Mr. Fee implies, a certain amount of personal equation is probably present in all observations, and this is not particularly objectionable provided it is constant and that proper methods are taken to eliminate the ill effects.

I have in mind the plan used by Major Bowie, Chief of the Division of Geodesy of the U.S. Coast and Geodetic Survey, in determining whether a prospective geodetic observer has a constant or an erratic personal equation. This applies especially to first-order triangulation work, where the object is centered midway between two parallel lines, but the test should be applicable to transit observers.

Any plain sheet of paper about 8 in. in width, with parallel sides, may be used. The prospective observer, placing himself squarely in front of the paper, quickly marks a point about one-half inch below the top of the sheet and midway, by estimation, between the two parallel sides. The upper portion of the sheet is then folded backward so as to obscure the first point, and a second dot is placed near the top in the same manner. The paper is again folded backward and the bisection process continued a number of times.

The paper is then unfolded and a straight line drawn along the mean position of the dots. The distance between this mean line and the true center line of the sheet indicates the constant personal equation, while the position of the dots, with respect to the mean line, shows whether there is an appreciable erratic tendency on the part of the observer.

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of Southern California

Beaumont, Calif.
August 8, 1931

Supply of Civil Engineers in Excess of Demand

Dear Sir: Whither engineers? Or—since I have been more in a position to observe in fair perspective the plight of the civil engineer—should I qualify the question and ask "Whither civil engineers?"

Were Horace Greeley living today, it is doubtful if he would again utter his famous advice, "Go West, young man, go West!" Engineering opportunity is becoming increasingly less a question of geographical location. The engineering schools in the Western States experience the same difficulty in placing their graduates that confronts the schools in the East. Some proclaim South America the fertile field of opportunity. However, this continent is now well supplied with American engineers in addition to the overflow of brilliantly trained men from the technical schools of Europe. Russia has offered some relief to a small number of American engineers, but the future development of the Soviet Republics is as yet a matter of speculation. Each year China sends a large number of her most intelligent youths to America for an engineering education, thus further limiting the prospective scope of our activities. And in all professions, England is depending to a great extent on Australia and India to support the overflow from her heavily populated island.

It seems to me that technical journals might render an invaluable service to the profession, if they could help to stem the great tide of civil engineers pouring continually into the field of practice. The greater the number of engineers who can be diverted into the fields of finance, industrial production, and sales, the better will be the future for all concerned. Perhaps if our engineering graduates could be apprised of the gravity of the situation and the dangers of an overcrowded profession they would temper their love for "design and construction," with intelligence.

From the inherent nature of engineering work in this age of exacting competition, the civil engineer must be, in a sense, an economist. He must, then, realize that the effect of flooding the engineering market with graduates in excess of the demand will inevitably tend to reduce their living status to that of the artisan, or even lower. A few graduates may hope to be successful as consulting engineers, but it has long been admitted that the day of the consulting engineer is waning. Generally, then, the outlook of the engineering graduate must be to a life of employment with the engineering department of large corporations or with the Government, municipal, or state bureaus.

With the present depression, there have arisen numerous tragic cases of engineers, well educated, intelligent, and capable, who through unemployment have been thrust upon charity—whether that of a relative, friend, or former employer. This professional problem can be corrected only by revision of the young engineers' aspirations to the purely technical positions. My viewpoint may be assailed as mercenary and ignorant of the spirit which inspires the engineer applying himself to the perfection of even a minute detail of a great engineering project. However, this attitude during times of depression often fails to feed our hungry families, nor does it necessarily advance the potentially capable members of our profession.

It is not my intention to discourage the youth entering the engineering schools, but rather to encourage recent graduates of these schools to seek new fields. A recent statement by Mortimer Elwyn Cooley, Dean Emeritus, College of Engineering, University of Michigan, calls attention to the fact that the engineer's greatest contribution to progress lies in the "habit of straight and honest thinking from cause to effect." This expression of opinion, coming from one renowned in the engineering profession, is certainly an adequate challenge to the graduate to enter fields where his services are demanded. I admit the attractions of the technical problems to be dealt with by the engineering departments of corporations and governments, but feel that other fields of activity offer many problems equally attractive.

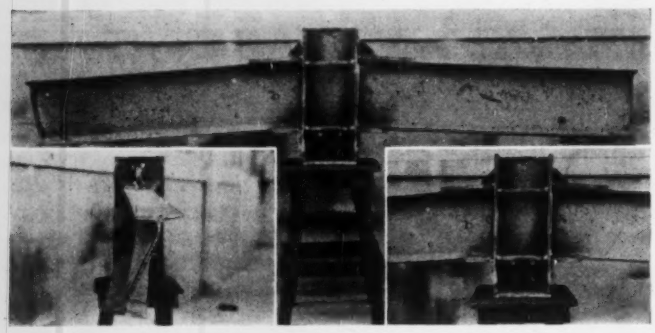
FRANK B. CAMPBELL, Jun. Am. Soc. C.E.
Junior Engineer, U.S. Bureau of Reclamation

Denver, Colo.
August 13, 1931

Experience with Welded Beams and Columns

DEAR SIR: It was with much interest that I read Mr. Jordan's letter, pertaining to the design of welded buildings, in the June issue of CIVIL ENGINEERING. The questions raised by Mr. Jordan are most pertinent, and I would like to submit for his information, and that of others interested, some of the welded details that were worked out in connection with our new Edison Building.

The new 13-story office building of the Southern California Edison Company, Ltd., has a content of nearly 4,000,000 cu. ft., and a gross floor area of 273,000 sq. ft. Since the building ordinances of Los Angeles do not permit an all-welded structure, it was decided to use ordinary riveted connections to carry all vertical loads and arc-welded connections to resist the earthquake forces for which the building was designed.



A WELDED CONNECTION TESTED TO FAILURE
Development of Bending Stress of 65,000 Lb. per Sq. In.

In the earthquake design the frame was calculated to withstand a horizontal force equal to one-tenth the acceleration due to gravity. This necessitated the design of girder and column connection to resist very large forces. After working out the design, we were convinced that arc welding was the only practicable method to use for these special connections.

In general, the floor system consists of a two-way slab, the bays being practically square. The beams, which are usually the same size, are framed into the center lines of the columns and are designed as continuous beams. The detail of the connection of the beam to the column was designed to develop full continuity.

The accompanying photographs show a sample connection that was made up and tested in order to determine whether or not it would be possible to develop the full strength of the beam in bending with this type of connection. This model, which was proportional in every way to the standard joint, failed at a bending stress of approximately 65,000 lb. per sq. in.

It will be noted that it was necessary to weld plates between the flanges of the "H" columns in order to transmit the stresses in the flanges of beam through the column, thus relieving a column flange of this horizontal load.

A great deal more could be said about the details of the welded connections that we adopted, but I think the one point that I have covered answers the question raised by Mr. Jordan.

H. L. DOOLITTLE, M. Am. Soc. C.E.
Chief Designing Engineer
Southern California Edison Company, Ltd.

Los Angeles, Calif.
August 9, 1931

Spier Falls Turbine Details

EDITOR: To amplify the construction details of the large turbine in the Spier Falls addition, described by Colonel Hogan in the March issue, under the title, "Record Hydro-Electric Turbine Installed," I should like to add several statements.

The complete turbine equipment and rack rake for this addition were furnished by the Newport News Shipbuilding and Dry Dock Company. In general, the turbine is of standard design, except for the influence of its large dimensions, which made it necessary, to a large extent, to sectionalize the parts, including the runner. The casing has an inlet diameter of 26 ft. 0 in. and measures 70 ft. over all. It is made up of 76 plates, ranging in thickness from $^7/_8$ to $^5/_8$ in., and formed to a smooth spiral shape.

The body of the runner is in three sections held together by two heavy cast-steel shrink bands. The runner was given a preliminary balance in the shop, the final balance being made in the field after the bands were installed and the temporary bolting lugs removed. Such runners are balanced on a hardened steel pin with spherical surface located on the vertical axis of the runner slightly above its center of gravity. The bands were electrically heated for installation. The draft tube is of a special elbow type in which the water passes around the bend in a comparatively thin sheet in order to avoid losses usually existing at this point. The regain takes place in the vertical and horizontal portion of the tube only.

The turbine gates were arranged so that their end clearance could be adjusted after installation. This resulted in the remarkably low leakage of about 7 sec-ft., or less than one-tenth of 1 per cent of the rated discharge. This is of considerable importance since the unit will be shut down a large portion of each day. The babbitted bearing is arranged so that the gland in the crown plate may be inspected and adjusted with the unit in service and so that any water leakage may be kept from getting into the oil sump.

R. V. TERRY
Hydraulic Engineer
Newport News Shipbuilding and Dry Dock Company

Newport News, Va.
July 21, 1931

Partially Supported Columns

Dear Sir: I have frequently met with a problem in structural design which is so recurrent that it has probably been discussed in some of the Society's publications. However, I have never been able to find any reference to the subject so thought that possibly the readers of Civil Engineering might be able to give me some information.

Briefly, the problem concerns the design of compression members fully supported at one point and partially supported at some intermediate point.

For example the lower tier of a column extending from the base of the column to the second-floor level is fully supported at the second floor, while the framing at the first floor is such that support is provided only for the web. The total load between the first and second floor is 350,000 lb. and the height is 20 ft. 0 in.; the total load from the base of the column to the first floor is 500,000 lb. and the height is 12 ft. 0 in. If the design of the column is controlled by the radius of gyration of the section about an axis perpendicular to the web, the question arises as to the load to be used. To use a loading of 500,000 lb. and an unsupported length of 32 ft. 0 in. (the total height of column to the second floor) would undoubtedly give a section of ample strength, but it is my belief that there would be a waste of metal, owing to the fact that the load of 500,000 lb. is imposed on the bottom 12 ft. 0 in. only. To design the section for a load of 350,000 lb. on the length of 32 ft. 0 in. would, of course, give a column of insufficient strength. Apparently the proper design lies between these two extreme conditions, but I do not know just where.

Any information on the subject will be deeply appreciated.

S. B. Downey, Assoc. M. Am. Soc. C.E.
Chief Draftsman, The M. A. Long Company

Baltimore, Md.
July 15, 1931

Virginia's Convict Labor Policy Criticized

Dear Sir: I was interested in reading the article on "Convict Labor in Highway Construction," in the June issue of Civil Engineering. But I do not agree with his justification of this labor on the grounds of economy and humanitarianism.

Highway work throughout the country furnishes the most feasible method of unemployment relief, and it therefore follows that, at this time, the employment of convicts on highways is a most deadly method of aggravating the unemployment situation. Virginia's policy in this respect has beaten down the wages of free labor and has had an effect on living standards not only in Virginia but in other states into which Virginia labor is forced to go because of low wages at home.

The author lays great stress on the benefits to the convict resulting from this labor. It is claimed that he learns highway work and is thus better qualified, upon release, to make his way. Such ex-convicts will stand little chance of getting a job in Virginia, for the convict labor policy of this state has largely eliminated road contractors with whom these ex-convicts might get employment. It is out of the question to expect that the 2,000 convicts who, he claims, are in Virginia convict road gangs, or even a sizeable fraction of them, would be able to get such employment. Their concentration in a single field makes it much more liable that they will remain unemployed and become "repeaters," than would be the case were the employment of prisoners diversified so as to more clearly correspond with the conditions that they will encounter when released. The employment by Virginia of unparalleled numbers of convicts on road work deprives free citizens of the opportunity of earning a livelihood and makes potential criminals out of many of them. Furthermore, on the average, each free laborer is supporting three others. Is the welfare of free citizens and their families less important to Virginia than the employment of her criminals?

Nor is the premise which attempts to show a saving to Virginia in the use of convicts valid. The value of any man's labor is in what he produces—not in any assumed earning power. The product of labor is entirely ignored and figures are based on the assumption that each convict earns $1.50 a day. According to this logic, all that any business man need do is put more men on his payroll and his profits will increase directly with the number of men employed. The only way to determine the profit or loss resulting from employment of convicts is to put the convict organizations on an equal competitive basis with competent contractors, find out at which price the job can be contracted, and keep account of all costs including that of equipment and overhead. It is advisable also to make allowance for the loss to the public in taxes which the private contractor pays—he pays about 18 different kinds of taxes, while the state pays none. When the job is finished, these figures should be published side by side. Virginia does not do this, so the purported saving is assumed rather than real.

However, waiving for the moment the false premise, the figures given cannot be substantiated. No one can successfully contend that a convict is worth as much as a free laborer. The convict probably cannot, under the best conditions, be made to show one-half the efficiency that a comptent contractor can show with free laborers. Therefore $1.50 a day for a convict, which is all that common labor is getting in Virginia today, is at least twice what it should be. Furthermore, the statement that the convicts work 250 days a year is out of line with reports by the U.S. Bureau of Public Roads that from 180 to 200 days are suitable for highway work in this section of the country. On this basis, if road convicts are, as the author asserts, costing the state $275 a year for support they are showing a per capita annual loss to the state of $125 to $150 apiece. The other item in the alleged saving, based on the cost of keeping convicts in prison ($240 a year) is much higher than in the states bordering on Virginia. For example, this figure for one state is $199 per prisoner per year, for another it is $208.50.

Forgetting for the moment the welfare of our free citizens and their prior rights to employment, I admit that it is advisable for convicts to be employed. However, I suggest that Virginia might well consider employment of convicts in more diversified lines, in occupations where they would be less likely to break down the living standards of free citizens and interfere with private business. Convicts could, for instance, be used in the reforestation of many areas of Virginia. There are large areas upon which taxes cannot be paid and which private capital cannot afford to develop. On such enterprises as this convicts could be worked with much less effect on the employment of free citizens.

Harry J. Kirk
Assistant Manager, Engineering Construction Divisions, The Associated General Contractors of America, Inc.

Washington, D.C.
July 24, 1931

SOCIETY AFFAIRS

Official and Semi-Official

Minnesota State Capitol—St. Paul

St. Paul Prepares for Fall Meeting

The history of St. Paul, where the Society will hold its Fall Meeting, is rich and varied. Probably the first white visitor to the site of the city was the Jesuit missionary, Father Louis Hennepin, who went there in 1680, and La Salle visited the locality two years later. When in 1805, President Jefferson sent Lieut. Zebulon M. Pike to take possession of the region, he bought most of the present site of St. Paul and the Fort Snelling Military Reservation from the Sioux Indians for 60 gallons of whiskey, to which Congress later added $2,000 in cash. It was in 1823 that the first steamboat made its way up the Mississippi and arrived at St. Paul's Landing. When the Territory of Minnesota was organized in 1849, this village of 32 houses was made its capital and named St. Paul. It was incorporated as a city five years later.

Twin City Lock and Dam

Lying at the confluence of the Minnesota and Mississippi rivers, the City of St. Paul and its sister city, Minneapolis, are surrounded by a countryside of unusual interest. The lake region, which is the source of the water supply of both cities, is immediately to the north. Altitude ranges from the extreme low-water level of the Mississippi River, at elevation 68, to the hills in the residential section, some of which reach a height of 1,000 ft.

Ramsey County, in which St. Paul is located, is noted throughout the United States for its fine gravel roads, and its beautiful drives will afford pleasure to all visitors. Fort Snelling, the U.S. Military Reservation, lies on the other side of the Mississippi River. Also located across the river, but only seven minutes from the hotel district, is St. Paul's airport, containing 260 acres, with paved runways 150 ft. wide.

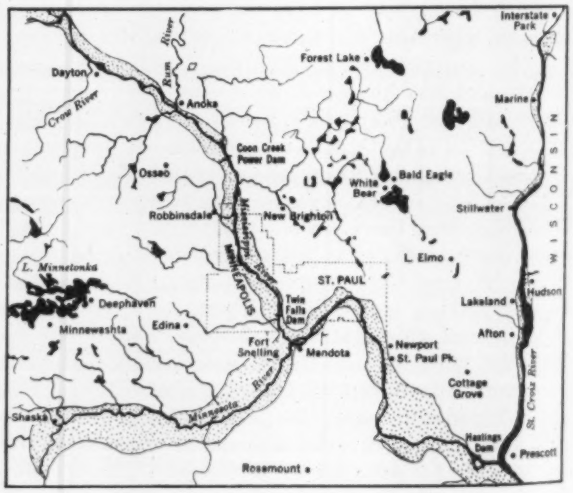
St. Paul covers an area of 55 sq. miles, of which more than 3 sq. miles are water. In 1928, the citizens approved a bond issue of $7,577,000 for a development program, and in 1929 the State Legislature gave Ramsey County the right to issue bonds to the amount of $8,000,000 for an improvement program within the city itself. The program includes a new City and County Building; substantial appropriations for schools, for enlargement of the airport, parks, and playgrounds; an addition to the Municipal Auditorium; a Public Safety Building; and a storehouse for the use of the Department of Public Works. Visiting engineers will have an opportunity to inspect the improvements now under way. The annual cost of the government of St. Paul is $12,000,000.

The Twin City Lock and Dam was built by the Government on the river between St. Paul and Minneapolis. The hydroelectric station at this point is operated by Henry Ford for the Twin City assembly and glass plant erected there. Four units operating with a maximum head of $35^1/_2$ ft. develop 18,000 hp. The present lock is 56 by 400 ft., and a second parallel lock is now under construction. Minneapolis is the head of navigation on the Mississippi.

At Hastings, 26 miles below St. Paul, an earth dam, 3,000 ft. long and a concrete structure containing twenty 20 by 30-ft. Tainter gates create a pool which extends 32 miles upstream to the Twin City Lock and Dam. This completes the canalization of the Mississippi River to the head of navigation. The lock has clear dimensions of 100 by 500 ft. However, the method of operation for navigation precludes the possibility of a power development at the Hastings Lock and Dam.

Meeting headquarters at the Hotel Lowry will be the focal point for numerous sight-seeing, inspection, and pleasure trips, planned to interest the ladies as well as their escorts. The Twin Cities, always beautiful, are exceptionally attractive in October. There will be a boat trip on the St. Croix River, with luncheon on board, and several drives, in the course of which members will have an opportunity to visit the University of Minnesota Campus; Lake Minnetonka; the Minnehaha Falls; the Mendota Bridge, one of the longest concrete arch bridges yet built; Fort Snelling; and the Lafayette Country Club.

A trip of extraordinary interest to engineers will begin at noon on Friday, October 9. The group will travel by train to Duluth, which is at the head of Lake Superior and the terminal of iron ore shipments. Here the visitors will be the guests of the Duluth Section of the Society. After the entertainment in Duluth, the party will entrain for Hibbing, the heart of the open-pit iron mining district—the Mesabi Iron Range—arriving in time for breakfast. Saturday morning is to be spent in an inspection of the Hull-Rust mine, the largest open pit mine in the world. The return trip to St. Paul will be routed again via Duluth, where the afternoon will be devoted to a sight-seeing trip about the Duluth and Superior harbors and an inspection of the ore docks. Thus will close what according to present indications will be a red-letter Meeting. The official program, with details of the papers to be



St. Paul and Environs

presented, and the entertainment to be provided for both members and guests, will be in the mail shortly.

The meeting will afford not only an occasion to add to one's knowledge of the latest in engineering developments of this region but also an unparalleled opportunity to renew old acquaintances and form new ones. The technical program is well filled; no time will be wasted; and both members and guests will be royally entertained.

An Impressive Technical Meeting

The technical papers delivered before the Sixty-first Annual Convention of the Society at Tacoma, Wash., July 8 and 9, make up this issue. In abstract form, the major points brought out by all the authors are emphasized. Limitations of space have prevented the inclusion of the many interesting discussions which formed an important part of the various sessions.

Quite obviously, it is impossible to convey one's reaction to the social attractions of such a meeting; but the essence of the technical program can be given. Although the necessity of condensing the wealth of material available is a handicap, yet an attempt has been made to do the utmost justice to the excellent papers within the necessary space limitations. The policy of the Society is to present this information to its membership immediately and in as generous a measure as an enlarged single issue permits.

Members are indebted to the various authors, to the local committees who worked so hard for the success of this meeting, and to the engineers who, by their attendance and loyal cooperation helped to make the Tacoma Convention notable in the annals of the Society. It is hoped that the abstracts here presented will convey in some measure the pleasure experienced by the many engineers who were privileged to be present in person when the complete papers were delivered.

First Ballot on Society Nominees for 1932

Official Nominees for Vice Presidents and Directors

New York
August 1, 1931

To the Secretary
American Society of Civil Engineers

The Tellers appointed to canvass the first ballot for official nominees report as follows:

Total number of ballots received		1,889
Deduct:		
Ballots from members in arrears of dues	101	
Ballots from members who have died since voting	2	
Ballots without signature	7	
Total not entitled to vote		110
Ballots canvassed		1,779

For Vice President, Zone 1

Arthur S. Tuttle	391
Scattering	121
Blank	35
Void	0
Total	547

For Vice President, Zone IV

D. C. Henny	184
Scattering	152
Blank	19
Void	1
Total	356

For Director, District 3

E. P. Lupfer	63
Scattering	12
Blank	8
Void	0
Total	83

For Director, District 5

John H. Gregory	134
Scattering	15
Blank	0
Void	0
Total	149

For Director, District 7

H. E. Riggs	165
Scattering	21
Blank	0
Void	0
Total	186

For Director, District 8

M. L. Enger	145
A. R. Lord	11*
Scattering	28
Blank	0
Void	0
Total	184

For Director, District 9

Robert Hoffmann	150
Scattering	7
Blank	0
Void	0
Total	157

For Director, District 12

J. C. Stevens	86
Scattering	11
Blank	4
Void	0
Total	101

For Director, District 16

E. B. Black	100
T. A. Leisen	36
R. C. Gowdy	34
Scattering	29
Blank	1
Void	0
Total	200

Respectfully submitted,
W. L. Cadwallader, *Chairman*

John W. Daly
C. W. Dunham
George E. Howe
Charles M. Madden

A. H. Morrill
Hans R. Jacobsen
A. H. Henckel
Philip Sander
Tellers

* *Mr. Lord has requested that his name be withdrawn.*

By the provisions of the Constitution of the Society, the first ballot for Official Nominees is for the purpose of nominating the candidates for office. All candidates receiving 5 per cent or more of the total number of votes cast for the respective offices become official nominees and their names are placed on the second ballot. This ballot is now being received, but only by members who reside in the Districts and/or Zones in which a Vice President or Director is to be elected. The second ballot will be canvassed October 15.

Appointments of Society Representatives

Francis Lee Stuart, President, Am. Soc. C.E., has been appointed one of the Society's representatives on the John Fritz Medal Board of Award for the four-year term, beginning October 1.

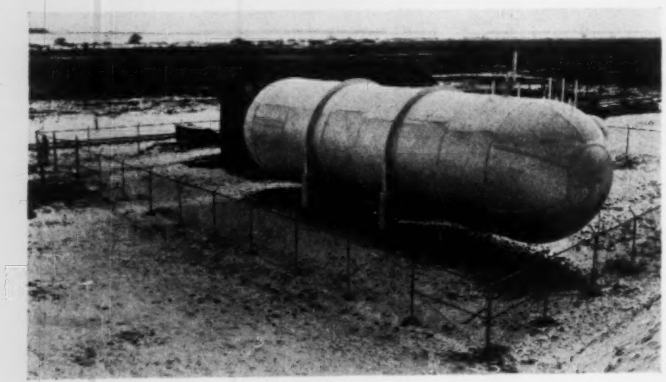
Verne LeRoy Havens, M. Am. Soc. C.E., has been appointed the Society's representative at the Fourth Pan-American Commercial Conference to be held in Washington, D.C., during the week, October 5 to 12.

A Preview of Proceedings

In September, PROCEEDINGS will carry three papers of considerable interest. The first, by Herman Schorer, Assoc. M. Am. Soc. C. E., on the design of large-diameter thin-shell pipe lines, discusses the use of ring girders at supporting points. The second, by Richard Hopkins, deals with the management of a contractor's organization. The third, by T. Warren Allen, M. Am. Soc. C.E., treats of the economies available to, but often overlooked by, highway contractors.

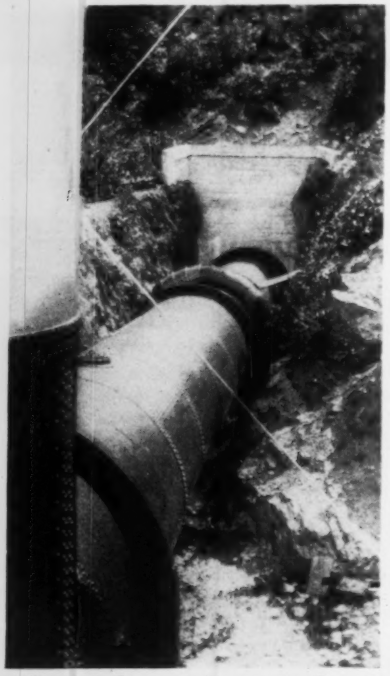
DESIGN OF LARGE PIPE LINES

In his paper, Herman Schorer, Assoc. M. Am. C.E., Chief Engineer for Thebo, Starr, and Anderton, San Francisco, applies the modern elastic theory of thin shells to the subject of designing large pipe lines. Such a pipe line, he argues, does not require any intermediate stiffening angles, even in the case of a comparatively thin shell. The theory advanced in the paper is based on the proposal to design ring girders entirely surrounding the pipe at supporting points instead of using the customary supporting saddles. Under such conditions it is found that the shell is mostly subject to direct beam and ring stresses, the load being transmitted to the supporting rings by the shear.



HIGH PRESSURE GAS HOLDER, KLAMATH FALLS, ORE.
Ring Girders Electrically Welded to Tank Shell

The basic equations upon which Mr. Schorer's subsequent derivation is based were introduced by Bauersfeld and by Finsterwalder, to whom reference is made in a German handbook published in 1928. Due credit is given to these and other European writers on this subject. The conclusions advanced for discussion by the membership are as follows:



GLINES CANYON PRESSURE PIPE
Showing Riveted Ring Girders and Expansion Joint (in Background)

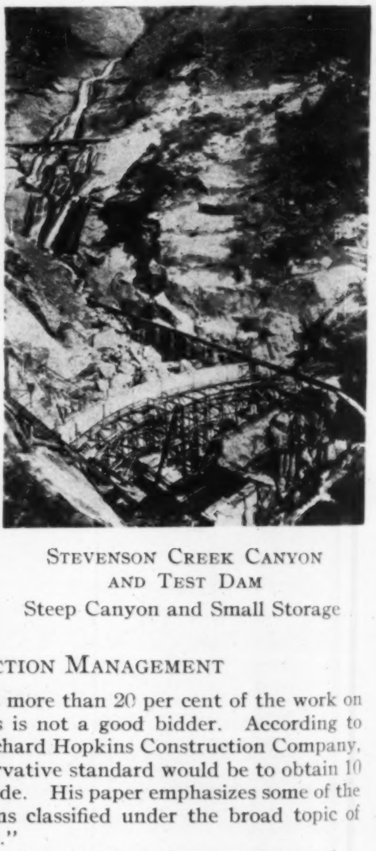
1. Cylindrical shells that are subject to water load may be designed most advantageously in accordance with the theory of thin shells of revolution. This theory permits the determination of an exact knowledge of the stress distribution, and leads to a safer and more economical design than the customary empirical method.
2. If the pipe is supported by disk-shaped members, the pipe walls are subject mostly to direct stresses, except for a very narrow zone of bending stresses at the support. For this reason the shell is entirely stable, without any intermediate stiffening ring, thus permitting the use of large spans in combination with thin plates.
3. Because of the reduced number of supports, the use of bearings with small friction values becomes feasible. In connection with expansion joints, it is possible practically to eliminate temperature stresses.
4. The larger spans permit easy access for painting, and in many cases the cost of excavation can be considerably reduced.
5. Because of the small deformation under all conditions of loading, the joints remain tight.
6. Pipe lines thus designed present a pleasing appearance because the finished structure expresses the typical properties of a thin shell.

HIGHWAY CONSTRUCTION MANAGEMENT

In giving a detailed description of the functions of the Division of Management, U.S. Bureau of Public Roads, Mr. Allen has also included data of interest to contractors regarding time-loss studies on highway construction work. His paper on "Highway Construction Management" should be especially valuable to contractors in pointing out sources of loss that are often ignored in actual practice.

Included are report forms such as those actually used on time studies by the Bureau of Public Roads, and the gathering and compilation of the necessary time-study data are illustrated in detail. Especially interesting is the diagram used to determine the required number of hauling units on concrete paving work. The bureau undertakes to assist contractors in increasing production on the highway work in which it comes in contact. To supply such graphs is part of this work.

On one job studied in Wisconsin the author found that the greatest cause of lost time was a shortage of hauling equipment. This was pointed out to the contractor early in the study, and as a result of the information furnished he realized the importance of supplying a sufficient number of hauling units for full mixer production. This is only one of the examples given to show the results of the research work done in this field by the Bureau of Public Roads.



STEVENSON CREEK CANYON AND TEST DAM
Steep Canyon and Small Storage

CONSTRUCTION MANAGEMENT

A contractor who obtains more than 20 per cent of the work on which he submits estimates is not a good bidder. According to Richard Hopkins, of the Richard Hopkins Construction Company, Albany, N.Y., a more conservative standard would be to obtain 10 per cent of the total bids made. His paper emphasizes some of the more important of the items classified under the broad topic of "Construction Management."

According to Mr. Hopkins, there is a right time and a wrong time to begin an active campaign for new jobs. For example, he states that there is no more pitiful figure in the entire construction world than that of the man who has a smoothly running and efficient organization on a particular job and who waits until a week or two before it is finished to start bidding on new work.

For the most part, this short and readable paper treats of the management involved in planning the actual construction work. The duty, the qualifications, and the personality required of the construction superintendent, of the foreman, and of the master mechanic are discussed in detail. Finally, the author opens up for discussion the subject of which practice is the better in maintaining construction schedules—that of making orderly progress along the line of a well planned schedule, or that of continually driving the organization to maintain a predetermined output capacity regardless of interfering conditions. Since there are advocates of each viewpoint, this phase is expected to draw the comments of many of our members who are familiar with construction work.

Short abstracts of papers presented by Mr. Hopkins and Mr. Allen before the Highway Division of the Society were published in Proceedings for March 1930. Because of their obvious merit and the importance of the subjects discussed, they are now printed in full, so that the membership of the Society can have an opportunity to discuss them. Needless to say, these papers have been revised and brought up-to-date by the authors.

Arch Dam Investigation

Perhaps the report of the Engineering Foundation's Committee on Arch Dam Investigation, published as Part III of the May 1928 Proceedings, establishes a record for long sustained discussion. After three years and three months of active discussion in Proceedings, this report comes to a close in September issue, with closing discussions from the following members of the Society: Fred A. Noetzli, answering discussors to Part 2; H. W. Dennis, to Part 3; Alfred D. Flinn, to Part 9; and W. A. Slater, giving an adequate answer to the discussors of his part of the report on the Stevenson Creek Dam.

News of Local Sections

Georgia Section

There were 45 members and guests present at the regular monthly meeting of the Section, held at the Atlanta Athletic Club on June 3. The meeting was addressed by J. B. Akers, Assistant to the Vice President in charge of maintenance, Southern Railway System, whose subject was "Railway Materials." Upon the conclusion of his address, B. L. Bugg, President of the A. B. and C. Railway, delivered an interesting talk on "Railway Economic Problems." His remarks included discussion of bus and auto competition, the use of the highway for private gain, the bus line franchise, and other topics of general interest.

San Francisco Section

A special meeting of the Section was held at the Engineers' Club, July 13, in honor of President Stuart, who came to San Francisco from the Tacoma Convention. There were 56 members and guests in attendance. The feature of the occasion was President Stuart's address on "The Engineer's Growing Civic Responsibilities," the full text of which he had delivered at the Convention.

Student Chapter News

Cooper Union Student Chapter

Members of the Cooper Union Student Chapter have enjoyed a number of gatherings during the past year, the attendance at each meeting averaging 125. Among the well known engineers who addressed the members upon different occasions were: Frank W. Skinner, Consulting Engineer; J. Charles Riedel, Assistant Engineer, Bureau of Sewers, Borough of Brooklyn, New York City; and Robert Ridgway, Chief Engineer, Board of Transportation, New York City.

Officers of the Chapter are as follows: J. Weinrub, President; H. Emery, Vice-President; W. J. O'Donnell, Secretary; and H. C. Wilson, Treasurer.

Manhattan College Student Chapter

Engineers and members of the faculty of Manhattan College Student Chapter addressed the members of the Chapter at their meetings held during the past year on a wide range of engineering subjects. The officers for the coming year have been elected as follows: Louis Persbacker, President; Victor Lo Pinto, Secretary; and Edward Ruddy, Treasurer. The position of vice president has not as yet been filled.

Pennsylvania State College Student Chapter

A summer camp for engineering work is conducted annually by Pennsylvania State College. The field work covers land, topographical, and engineering surveying, which includes stream gaging, weir observations and computations, and railroad and highway surveying. The Student Chapter of Pennsylvania State College has been prominently identified with the activities of this engineering session during the past summer. To encourage interest in the profession it offered prizes for the best talks given by students on engineering topics.

University of Dayton Student Chapter

Among those who addressed the University of Dayton Student Chapter during the past year were: Mr. Brewster, of the National Lumber Manufacturers' Association, Washington, D.C., whose subject was the source and treatment of lumber; Mr. Riddle, Civil Engineer of Dayton, Ohio, who talked on "Bridges and the World War"; and Maxwell C. Dice, of Dayton, who spoke on the benefits and validity of patents as applied to engineering processes. During the course of the year the Chapter, as a body, made inspection trips to various points of engineering interest in the State of Ohio.

University of Oklahoma (Stadia Club) Student Chapter

The April issue of *The Sooner Magazine*, published monthly by the alumni of the University of Oklahoma, was devoted to the engineers, both graduate and undergraduate, of the university. Foremost in the undergraduate engineering life of the university is the Stadia Club, the local Student Chapter. Since its organization in 1922, the club has increased steadily in membership, and the past year has been one of unusual activity for it.

University of Pennsylvania Student Chapter

Much of the activity of the University of Pennsylvania Student Chapter has, during the past year, taken the form of inspection trips to the various engineering projects that are now in process of construction in Philadelphia and its vicinity. These trips included visits to the U.S. Bureau of Standards and the U.S. Bureau of Public Roads, in Washington, and an extensive inspection of the Curtis Bay coal-handling plant of the Baltimore and Ohio Railroad and of the hydro-electric power plant at Conowingo. Among those who addressed the members at their meetings were S. Schwaab, Consulting Engineer on the Pennsylvania Tube under the East River; and Carl Janson, Engineer of the Dravo Construction Company.

During May it was announced that J. C. Iandolo, a member of the Chapter, has been awarded the second prize for a bridge design, given each year by the American Institute of Steel Construction.

A Year's Mail

Records of the Society's Mailing Department, which handles all material for the post offices, cast some light on the amount of work in correspondence and publication carried on during 1930. An exact count gives the astounding total of 598,597 pieces of mail which were sent out during the year, at a total cost to the Society of $14,836.86.

Probably every member has received first-class mail from the Society bearing no stamp but instead red indicia, that is, distinctive marks of the Government-controlled stamping machine. In all, 245,266 pieces of mail of this character were sent out. Included also in the year's business were 14,851 volumes of Transactions and about 149,000 copies of Proceedings. During the last three months, Civil Engineering accounted for about 46,300 copies. For Technical Divisions alone, 31,400 pieces of mail were sent out.

Evidently the Mailing Department as yet has not heard of any depression.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for October

Damage to crops, reduction of water power, and pollution of domestic water have resulted from the present period of prolonged drought and hot weather. It seems pertinent to inquire how widespread these effects have been, just how serious the situation is as compared with previous periods of subnormal precipitation, and what will be the after-effects. For the October issue of Civil Engineering, John C. Hoyt, M. Am. Soc. C.E., Hydraulic Engineer in charge of the Surface Water Division of the U.S. Geological Survey, has prepared a comprehensive article on these much discussed questions.

Handbooks on civil engineering give information largely gathered from the temperate zones. The engineer who operates in the tropics encounters problems for which experience in cooler regions provides no solutions. In an article by Carl B. Andrews, Assoc. M. Am. Soc. C.E., Professor at the University of Hawaii, the engineering and construction problems that confront the engineer in the tropics are set forth. For example, the characteristics of laterite, a clayey soil peculiar to the tropics, are being learned through hard experience with heaved pavements and settled walls built upon it. Another plentiful material is coral rock, widely used for buildings, walls, and road metal.

An illuminating and instructive article to appear in the October issue of Civil Engineering, entitled "Burned Clay Masonry," is by F. E. Emery, M. Am. Soc. C.E., Engineer of the Structural Clay Tile Association. Clay has been used by man since earliest times, yet new uses are continually being found for it. Engineers today employ it in fireproofing, conduits, pipes, tile walls, roofing, and in ornamentation. Mr. Emery discusses methods of making hollow tile walls storm-proof and gives illustrations of effective construction.

So many interests are involved in the planning, financing, and construction of any grade-crossing elimination that negotiations are often long drawn out. From his own experience, H. A. Shuptrine, Assoc. M. Am. Soc. C.E., Bridge Engineer of the Board of Wayne County Road Commissioners, presents methods of effectively bringing such negotiations to a satisfactory and rapid close. He believes that a grade separation job is half done when it is released to the draftsmen for detailed plans.

Other articles dealing with the problems of transportation and railroad terminal location will be included, together with readers' discussions of articles which have already appeared, and the presentation of news concerning the activities of the Society. The October issue will mark the first anniversary of the establishment of Civil Engineering as the youngest of the Society's publications.

Oil Pollution

Pollution of our public beaches by oil is still an unsolved problem. It has its international as well as its national aspects for, wherever jurisdictions overlap, pollution is particularly difficult to control. A recent report of the Joint Committee on Oil Pollution of the American Engineering Council indicates that pollution is declining. Believing that there is a higher degree of efficiency in state or local jurisdiction in the matter of oil pollution, this committee, of which Robert S. Weston, M. Am. Soc. C.E., is Chairman, and Abel Wolman, M. Am. Soc. C.E., Secretary, opposes pending legislation to extend Federal jurisdiction over waters which are now under state control.

It is pointed out that the committee has frequently been confronted with the fact that one of the principal offenders in this connection is the U.S. Navy, which, according to reports, discharges oil from oil-burning warships without much regard for either state or Federal rules.

COMING EVENTS

St. Paul in October

Fall Meeting of the
American Society of Civil
Engineers

St. Paul, Minn.

October 7, 8, 9, 10, 1931

American Association of Engineers
Annual convention at Huntington, W.Va., September 28–30

American Association of State Highway Officials
Annual meeting at Salt Lake City September 28—October 1

American Public Health Association
Annual Convention at Montreal, September 14–17

American Society of Mechanical Engineers
Regional meeting in Kansas City, Mo., September 7–10
Iron and Steel and Institute of Metals Division
Meeting in Boston, September 21–24

American Welding Society
Fall meeting at Boston, September 21–25

International Association of Public Works Officials
Conference in New York City, September 24–26

New England Water Works Association
Annual convention in Boston, September 29–October 2

First Engineering Societies Monograph Announced

Many interesting and important manuscripts, which become available to the four Founder Societies, cannot be printed by the societies individually. From time to time, a committee composed of members of each of these societies will, in cooperation with a book publisher, select certain monographs for publication. Members of the societies are enabled to benefit by a preferential rate, and the Engineering Societies Library shares in the financial success of the venture.

For the first of these monographs the Engineering Societies Monographs Committee announces the publication this month of *Plasticity*, by Dr. A. Nadai, of the Research Laboratories of the Westinghouse Electric and Manufacturing Company. Dr. Nadai was formerly Professor of Applied Mechanics at the University of Goettingen, Germany.

This book, based on his original German work, but amplified and enlarged by the addition of new information, brings together the observations of engineers, metallurgists, and physicists regarding the plastic deformations of metals and then summarizes the laws which are available for the more exact prediction of the distribution of stress. Also, it discusses the fundamentals of the theory of plastic flow in materials, especially in the metals.

It is to be published for the Engineering Societies Monographs Committee by the McGraw-Hill Book Company, Inc., 370 Seventh Avenue, New York, N.Y. The price on publication will be $5.00, but there will be a special pre-publication price of $4.00 to members of the four national engineering societies.

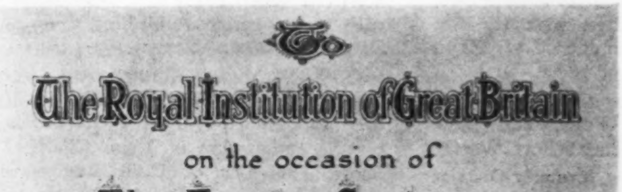
Faraday Centenary Celebration, 1831–1931

In 1791, Michael Faraday, the son of an English blacksmith, was born at Newington Butts, Southwark, London. As a journeyman bookbinder he developed an interest in the writings of Sir Humphry Davy and later became his assistant. In 1827 he succeeded Sir Humphry in the Chair of Chemistry, Royal Institution. His researches in this science are noteworthy but the great additions made by him to the range of human knowledge were mostly in the related sciences of magnetism and electricity. In 1831 he discovered electromagnetic induction. This and subsequent discoveries have given him the significant appellation, "the creator of the age of electric power." He died in 1867.

In 1927, with donations and the proceeds of a foundation fund, widely contributed to by individuals as well as by scientific and technical organizations, the Society among them, the Metropolitan Borough of Southwark established the Faraday Memorial Library, for the free use and benefit of the public. Its

Michael Faraday, 1791–1867

purpose is to honor and keep alive the memory of Michael Faraday. It is of interest to note the manner in which a municipality has thus honored a great scientist. This year is the centennial of Faraday's monumental discovery.



To

The Royal Institution of Great Britain

on the occasion of

The Faraday Centenary

1831 1931

The American Society of Civil Engineers

sends greetings - appreciating the fundamental significance of the principles of Electro-Magnetic Induction - and desiring to join with the Royal Institution in showing honor to the great scientist who made this epochal discovery one hundred years ago.

Francis Lee Stuart
President

George T. Seabury
Secretary

Presented to the Royal Institution of Great Britain

Brunel and His Son Build the Thames Tunnel

Several of the early attempts to connect the two sides of the Thames River by tunneling through the soft mud of the river bed resulted in failure. In 1823, Mark I. Brunel began plans for the celebrated tunnel to connect the bottom of shafts sunk 72 ft. deep on the opposite shores at Wapping and at Rotherhithe. The plan of construction of the 1,200-ft. tunnel was to excavate with a shield followed immediately with brick lining. Without the benefit of compressed air to hold up the very unstable clay and mud, Mr. Brunel's shield was ingeniously designed.

Being 36 ft. wide, 22 ft. high, and, 9 ft. long over all, it consisted of 12 similar vertical cast- and wrought-iron frames 22 ft. high and 3 ft. wide. The 12 frames may be compared to 12 giant books side by side. Frames could be advanced a short distance independently of one another, yet each gave lateral support to the others. They were divided into three cells, there being 36 in all, each large enough to hold a workman. Each frame had an iron roof extending back to the brickwork lining and a pair of jack screws at the top and bottom abutting against the front end of the brickwork, to push it forward. Similarly, on the sides, iron plates extended back from the face of the excavation to the brickwork.

The face of the excavation was held by a range of 3 by 6-in. poling boards 3 ft. long, placed across the face of the frames. Each poling board was arranged horizontally and pressed against the earth by two small jack screws supported by the opposite edges of the frame. These poling boards, more than 1,000 in number, covered the tunnel face in front of the frames.

In the process of excavation, the miners, beginning at the top, took down one, or at most, two of the poling boards, excavated the material in front for a distance of 3 in., replaced the poling boards and, with the base of the small jacks resting on the adjacent frames, pressed the boards back against the new face. When all the poling boards of one section had been thus advanced 3 in. and were supported by the adjacent sections, the section itself was advanced to the face by the section jacks. After each of the 12 sections had been tediously advanced in this manner, another ring of the brickwork was built up. Brunel called this shield an ambulating cofferdam traveling horizontally.

The brickwork was built in a rectangular mass with outside dimensions the same as those of the shield. The tunnel consisted of two parallel horseshoe arched roadways 14 ft. wide and 17 ft. high, connected at intervals with small cross arches. The thickness of the brickwork at the crown and invert was $2^1/_2$ ft.; at the sides, 3 ft.; and in the central wall, $3^1/_2$ ft.

Construction of the shafts began in March 1825 and by November the shield had been erected at the bottom of the Rotherhithe shaft, and progress on the tunnel began. During the following year about 400 ft. of progress was made and visitors were being admitted to the works. However, both Mr. Brunel and his son, Isambard K. Brunel, who had become resident engineer, felt constant apprehension of danger to visitors for fear the Thames would break in. A number of small breaks had occurred which justified this apprehension.

On May 18, 1827, the Thames did break in just after the work had been visited by Lady Raffles and a considerable party. The tunnel and shaft were filled with water. The shield had pierced into a dredged spot. After bags of clay had been dropped to the bottom of the Thames over the frames and allowed to consolidate, the water was lowered sufficiently so that an examination of the shield and heading could be made by rowing a boat from the shaft to the frames. On June 17, a party of visitors was reluctantly taken into the heading by rowboat. On this occasion an amusing incident occurred. Isambard Brunel "was exceedingly unwilling to permit his visitors to make this expedition into the arch; but on the assurance that they could all swim perfectly well, he consented to take them, with the understanding that, if he jumped overboard, they were immediately to follow his example, and swim after him to the shaft. While they were in the arch, Mr. Brunel [Isambard] fell overboard. As soon as he recovered himself, and turned to swim back to the boat, he remembered that he had unwittingly given to his companions the signal to jump out into the water. He was much amused, on looking up, to see that they were not swimming after him, but were still sitting in the boat clinging to the gunwale, with faces expressive of blank despair."

On January 1, 1828, when 600 ft. of tunnel had been completed, another break occurred which caused financial difficulties and cessation of the work for seven years. Isambard Brunel reported this break as follows: "I had been in the frames (shield) with the workmen throughout the whole night, having taken my station there at ten o'clock. During the workings through the night, no symptoms of insecurity appeared. At six o'clock this morning (the usual time for shifting the men) a fresh set or shift of the men came on to work. We began to work the ground at the west top corner of the frame: the tide had just then begun to flow; and finding the ground tolerably quiet, we proceeded by beginning at the top, and had worked about a foot downward, when on exposing the next six inches, the ground swelled suddenly, and a large quantity burst through the opening thus made. This was followed instantly by a large body of water. The rush was so violent as to force the man on the spot, where the burst took place, out of the frame (or cell) on to the timber stage behind the frames. I was in the frame with the man, but upon the rush of the water I went into the next box (or cell) in order to command a better view of the irruption, and seeing that there was no possibility of then opposing the water, I ordered all the men in the frames to retire. All were retiring, except the three men who were with me, and they retreated with me. I did not leave the stage until those three were down the ladder of the frames, when they and I proceeded about twenty feet along the west arch of the Tunnel. At this moment the agitation of the air, by the rush of water, was such as to extinguish all the lights and the water had gained the height of our waists. I was at that moment giving directions to the three men, in what manner they ought to proceed in the dark to effect their escape, when they and I were knocked down, and covered with a part of the timber stage. I struggled under water for some time, and at length extricated myself from the stage, and by swimming and being forced by the water, I gained the eastern arch where I got a better footing, and was enabled by laying hold of the railway rope, to pause a little, in the hope of encouraging the men who had been knocked down at the same time with myself. This I endeavored to do by calling to them. Before I reached the shaft the water had risen so rapidly that I was out of my depth, and therefore swam to the visitor's stairs, the stairs for the workmen being occupied by those who had so far escaped. My knee was so injured by the timber stage that I could scarcely swim, or get up the stairs, but the rush of the water carried me up the shaft. The three men who had been knocked down with me were unable to extricate themselves, and I am grieved to say, they are lost; and I believe also two old men, and one young man, in other parts of the work."

Work was not resumed on the tunnel until 1835, when a government loan enabled its continuation. A new form of shield was substituted and after three more irruptions of the river, the tunnel was completed in March 1843 and opened to the public 18 years after it was begun. Although built for a highway tunnel it never paid and was finally sold to the East London Railroad, and trains operated through it. For his distinguished work the elder Brunel was knighted in 1841.

Extracts and facts from "Life of I. K. Brunel, Civil Engineer," Longmans, Green and Company, London, 1870.

NEWS OF ENGINEERS

From Correspondence and Society Files

R. P. Forsberg, previously Principal Assistant Engineer of the Pittsburgh and Lake Erie Railroad, has been appointed Chief Engineer of the company.

William H. Barton, Jr., head of the Department of Civil Engineering at Pennsylvania Military College, has been with the U.S. Bureau of Public Roads for the past summer as a specialist in highway research.

William Wayne Anderson, formerly Division Engineer, State Highway Department, is now in contracting and private engineering work in Medina, Ohio.

J. S. Sawyer, of the Shell Oil Company, San Francisco, Calif., has been transferred to an associated company in New York City.

E. G. Larson is with the International Railroad of Central America at Guatemala City, Guatemala.

Porter W. McDonnell has organized Toledo Surveyors, Inc., of which he is President and General Manager. Until recently Mr. McDonnel has been in the employ of Waddell and Hardesty, Consulting Engineers, New York City, on the construction of the Maumee River high-level suspension bridge at Toledo.

J. H. Meursinge, formerly of Larkspur, Calif., is now connected with the Six Companies, Inc., at Boulder City, Nev., as an Assistant Engineer.

George D. Fairtrace, who was City Manager at Wichita Falls, Tex., now holds the same position at Fort Worth.

Russell E. Snowden, formerly District Engineer, State Highway Commission, at Kinston, N.C., is at present at Snowden, N. C., where he plans to engage in practice as a consulting engineer.

A. K. Morgan, now Acting Superintendent of Jones Beach State Park, was prior to this, Assistant to the Architectural Engineer, Long Island State Park Commission.

Arthur Schultheis, is now associated with the Trojan Engineering Corporation at 40 Exchange Place, New York City. He was formerly with the Public Works Engineering Corporation of the same address.

Carl G. Paulsen, formerly District Engineer, U.S. Geological Survey, with headquarters in Boise, Idaho, has been transferred to Washington, D.C., to take the position of Chief of the Surface Water Division of the Geological Survey.

Fred E. Swineford, previously Director of Public Service, City of Akron, is now Chief Engineer of the Ohio Crushed Stone Association, at Columbus, Ohio.

F. E. Lawrence, until recently an engineer in Groveland, Mass., has accepted a position with the Century Wood Preserving Company in Charleston, S.C.

Howard E. Boardman has been appointed Dudley Professor of Railroading at Yale University. He was formerly Construction Engineer for the Boston and Maine Railroad with headquarters in Boston.

Gerald W. Knight, Consulting Engineer, has been appointed one of the New Jersey Commissioners of the Tri-State Treaty Commission for the abatement of the pollution of the waters adjacent to New York. This commission, of which Mr. Knight has been made secretary, is composed of appointees from the states of Connecticut, New York, and New Jersey.

Thomas R. Newell has been promoted to the position of District Engineer for the U.S. Geological Survey at Boise, Idaho.

John S. Butler, Major Corps of Engineers, U.S.A., will serve as Corps Area Engineer for the 7th Corps Area, at Omaha, Neb. He has for the past four years been District Engineer for rivers and harbors at Seattle.

Walter B. E. Anthony has joined the U.S. Engineer Corps and is now Resident Engineer in charge of the Brandon Road lock and dam. Formerly Mr. Anthony was a Resident Engineer with the Detroit Water Board.

Edward J. Mehren, Vice President of the McGraw-Hill Publishing Company, has been elected President of the Portland Cement Association.

Norman M. Stineman, who was a Structural Engineer for the Portland Cement Association of Chicago, has accepted the editorship of *Concrete,* with the Concrete Publishing Company, also in Chicago.

C. T. Schwarze, formerly an Associate Professor at New York University, has been appointed to fill the vacancy caused by the resignation of Dean Charles H. Snow, Professor of Civil Engineering.

Dwight Redman has been placed in charge of the disbursing office of the U.S. Indian Irrigation Service at Wapato, Wash. This office has been removed from Yakima where L. M. Holt was in charge. Recently, Mr. Holt was transferred to Salt Lake City, Utah, as Supervisor of Projects in Washington, Oregon, Utah, and Nevada.

R. C. Marshall, Jr., is Vice President in Charge, of the New York office of the B-W Construction Company at 101 Park Avenue. He was formerly President of the Sumner Sollitt Company, of Chicago, Ill.

Robert Vance Orbison, who for the past six years has been City Manager of Pasadena, Calif., has submitted his resignation to the Board of Directors.

H. M. Shepard, lately Assistant Engineer for the Erie Railroad, New York City, is now an Assistant Chief Draftsman for the same railroad, with headquarters in Cleveland, Ohio.

D. H. Sawyer, recently Secretary, Heating, Board of Trade, New York, has been appointed Director, Federal Employment Stabilization Board, Washington, D.C.

Herbert W. Yeo, who was with the General Land Office in Denver, is now with the International Water Commission of El Paso, Tex.

Robert J. Newell, Superintendent for the U.S. Bureau of Reclamation on the Boise Project, has been transferred to the Yakima Project where he will have charge of construction of the Cle Elum Dam.

George H. Miller, formerly Structural Engineer for the City of Tacoma, Wash., is now on the staff of the City Engineer of Wenatchee, Wash.

Paul P. Rice is an Instructor in Civil Engineering and Land Surveying at Lafayette College, Easton, Pa.

S. H. McCrory, formerly Chief Engineer of the Division of Agricultural Engineering, of the U.S. Bureau of Public Roads, Department of Agriculture, has been appointed Chief of the new Bureau of Agricultural Engineering.

Asa C. Baldwin is the Northwestern representative for Schlumberger Electrical Prospecting Methods.

Nat H. Neff, formerly Superintendent of Highways, Orange County, Santa Ana, Calif., has been appointed City Engineer of Huntington Beach.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From July 10 to August 7, 1931

Additions to Membership

Alexander, John Billings (Assoc. M. '31), Associate Hydr. Engr., U.S. Engrs., 606 Safety Bldg., Rock Island, Ill.

Couse, Walter Learned (Assoc. M. '31), Engr. and Supt. in Chg. of Constr., W. E. Wood Co., 2714 Union Guardian Bldg. (Res., 11 McLean Ave.), Detroit, Mich.

Cutter, Alfred Breen (M. '31), City Mgr., Cape May, N.J.

Davis, Orin Levis (M. '31), Mgr., Concrete Eng. Co., 505 Walsix Bldg., Kansas City, Mo.

Dayton, Cedric Lodge (Assoc. M. '31), Designer, Concrete Steel Co., 2 Park Ave., New York (Res., 16 Hollywood Ave., Tuckahoe), N.Y.

Decombe, Alberto (M. '31), Director of Reclamation, Dept. of Public Works, Republic of Chile; Prof. of Railways and Highways, Catholic Univ. of Santiago (Res., Avenida El Bosque 304— Los Leones), Sanitago, Chile.

de Westfelt, Gerard Philip (Jun. '31), 315 East 68th St., New York, N.Y.

DuBois, Charles Merwin (Assoc. M. '31), Engr., Allied Engrs., Inc., 709 Broadway, Piqua, Ohio.

Eisenhuth, Harold Paul (Jun. '31), Junior Engr., U.S. Geological Survey, 403 P. O. Bldg., Denver, Colo.

Fernstrom, John Archie (Jun. '31), 2020 Brown St., Napa, Calif.

Gahagan, Frederick Mussen (Assoc. M. '31), Pres., Hewlett Point Corp.; Vice Pres., Gahagan Constr. Corp.; Vice-Pres., W. H. Gahagan, Inc., 147 Remsen St. (Res., 19 Prospect Park, West), Brooklyn, N.Y.

Goerner, Ernest William (Assoc. M. '31), Eng. Draftsman, N.Y.C.R.R., 466 Lexington Ave., Room 932, New York, N.Y. (Res., 157 Linden Ave., Glen Ridge, N.J.).

Growdon, James Paul (M. '31), Asst. Chf. Hydr. Engr., Aluminum Co. of America, 2400 Oliver Bldg., Pittsburgh, Pa.

Guesmer, George Otto (Assoc. M. '31), Asst. Engr., U.S. Engr. Office, St. Paul (Res., 3011 Third St., North, Minneapolis), Minn.

Hagy, Ernest August (Assoc. M. '31), Cons. Engr., Box 588, Cincinnati, Ohio.

Haight, Frank Joshua (Assoc. M. '31), Tidal Mathematician, Div. of Tides and Currents, U.S. Coast and Geodetic Survey (Res., 5507 Thirty-third St., N.W.), Washington, D.C.

Hewitt, Leland Hazelton (Assoc. M. '31), Lieut., C. of E., U.S.A., Room 1068, New Navy Bldg., Washington, D.C.

Hjorth, Lawrence Rasmus (M. '31), Designing Engr., H. G. Balcom, 10 East 47th Street, New York, N.Y.

Holroyd, Normond Scripture (Assoc. M. '31), Engr. in Chg. of Constr., Solomon & Keis, 257 Broadway, Troy, N.Y.

Horovitz, Oscar Henry (Assoc. M. '31), Engr. and Sub-Contr., 17 Oldfields St., Dorchester, Mass.

Howe, Charles Shedd (Assoc. M. '31), Vice Pres., Los Angeles Testing Laboratory, 1300 South Los Angeles St., Los Angeles, Calif.

Kistler, Homer King (M. '31), Assoc. Prof., Hydr. Eng., The Pennsylvania State Coll. (Res., 510 West Fairmount Ave.), State College, Pa.

McGuinness, William James (Jun. '31), with Marc Eidlitz & Son, Inc., New York (Res., 31 Victor Pl., Elmhurst), N.Y.

Mallory, James Colvin (Assoc. M. '31), Supt. of Constr., Triest & Earle, Inc., Philadelphia (Res., 136 North Elm Ave., Aldan), Pa.

Maynard, Robert Lee (Assoc. M. '31), City Engr., City Hall, Asheville, N.C.

Moffatt, John Gray (Jun. '31), Asst. Engr., Truscon Steel Co., West Linn, Ore.

Morris, Theodore (Jun. '31), Assistant Engr., Water Service Dept., P.R.R., 609 Broad St. Station (Res., 6251 Wister St.), Philadelphia, Pa.

Mortenson, Myron (Jun. '31), 1133 Forest Ave., Ann Arbor, Mich.

Northrup, Donald Otto (Jun. '31), Asst. Engr., N.Y.C. & St. L. R.R. (Res., 1791 Delmont Ave.), East Cleveland, Ohio.

Pedgrift, Delmore George (Jun. '31), Estimator and Draftsman, James H. Pedgrift (Res., 4110 Broadway), Oakland, Calif.

Price, Keith Hargreaves (Jun. '31), 1423 Sturt St., Ballarat, Victoria, Australia.

Rosa, Joseph John (Jun. '31), 91 Barnes St. Extension, Waterbury, Conn.

Salazar Quezada, Jose Francisco (Assoc. M. '31) (J. F. Salazar & Co.), P. O. Box 1225, San José, Costa Rica.

Shafer, Paul C. (Assoc. M. '31), City Engr., City Bldg., Ravenna, Ohio.

Shah, Dhirajlal Somchand (Jun. '31), Morvi, Kathiawar, India.

Simoes, Carlos Quirino (M. '31), Acting Director, State Highway Dept., Rua Conselheiro Brotero 177, Sao Paulo, Brazil.

Stewart, George Thomas (M. '31), 7650 Colfax Ave., Chicago, Ill.

Thoroughgood, Robert William (M. '31), Prof., Civ. Eng., Univ. of Delaware (Res., 46 East Delaware Ave.), Newark, Del.

Toma, Jameel Shammas (Jun. '31), Asst. Engr., Bagdad-Iraq Ry., Bagdad, Iraq.

Van Camp, Paul Milton (Assoc. M. '31), 250 Locust St., Meadville, Pa.

Van Loan, William Sherman (Assoc. M. '31), Superv. of Constr., Board of Education (Res., 202 Summit Ave.), Buffalo, N.Y.

Vitz, Alvin (Jun. '31), 230 Parkway Ave., Hartwell, Cincinnati, Ohio.

Watts, Aubrey Byron (Assoc. M. '31), Fredericktown, Mo.

Young, Edmund Rufus (Jun. '31), Care, County Engr., Fort Bend County, Richmond, Tex.

Membership Transfers

Beyer, Adam Carl (Jun. '25; Assoc. M. '31), Dist. Mgr., Wallace & Tiernan Co., 7 Front St., San Francisco (Res., 2723 Benvenue Ave., Berkeley), Calif.

Campbell, Harry Lucas (Assoc. M. '20; M. '31), City Engr. (Res., 1550 Lewis St.), Charleston, W.Va.

Dunn, George Perry (Jun. '29; Assoc. M. '31), Asst. Engr., New York Telephone Co., 257 West Castle St., Syracuse, N.Y.

Goyette, Ernest Fred (Assoc. M. '21; M. '31), Specification Writer and Supt. of Constr., Allen & Collins, 75 Newbury St., Boston (Res., 326 Charles River Rd., Watertown), Mass.

Halsey, Wallace Haynes (Jun. '08; Assoc. M. '13; M. '31), 63 Main St., Southampton, N.Y.

Hardin, Eugene Albert (Assoc. M. '26; M. '31), Designing Engr., Dept. of Water Supply (Res., 13,574 Mendota Ave.), Detroit, Mich.

Haseley, Paul Fred (Jun. '28; Assoc. M. '31), Asst. Engr., Niagara Reservation Comm. (Res., 135 Sixty-eighth St.), Niagara Falls, N.Y.

Jordan, Robert Dawson (Assoc. M. '28; M. '31), Prin. Asst. Engr., State Highway Dept. (Res., 108 Magnolia Curve), Montgomery, Ala.

Kimberly, Clifford Bailey (Assoc. M. '28; M. '31), Dist. Engr., U.S. Fidelity & Guaranty Co. of Baltimore, Md., Midland Bldg., Kansas City, Mo.

Leclercq, Emile Paul (Jun. '26; Assoc. M. '31), Structural Designer, McClave & McClave, 600 Gorge Rd., Cliffside (Res., 1025 Palisade Ave., Palisade), N.J.

Marr, John Gentle (Jun. '28; Assoc. M. '31), Care, Bartholomew & Associates, 317 North 11th St., St. Louis, Mo.

Mathews, James Thomas (Assoc. M. '28; M. '31), Lt.-Commander, C.E.C., U.S.N., Public Works Officer, Navy Yard and 6th Naval Dist., Charleston, S.C.

Moore, Junius Teetzel (Jun. '20; Assoc. M. '24; M. '31), Pres., Fireproof Products Co., Box 1327, Charleston, W.Va.

Muhs, Fred Ross, Jr. (June '26; Assoc. M. '31), Care, Pottenger Sanatorium, Monrovia, Calif.

Olson, Kenneth Miles (Jun. '26; Assoc. M. '31), 449 East 81st St., Chicago, Ill.

Praeger, Emil (Jun. '15; Assoc. M. '21; M. '31), Cons. Engr., 307 West 49th St., New York (Res., 152 Stratford Rd., Brooklyn), N.Y.

Richards, Nathaniel Atherton (Assoc. M. '15; M. '31), Vice-Pres., and Chf. Engr., Purdy & Henderson, 45 East 17th St., New York, N.Y.

Schwind, Raymond Charles (Assoc. M. '25; M. '31), Engr., Ferber Constr. Co., 16 Johnson Ave., Hackensack (Res., 52 Spruce Ave., Ridgefield Park), N.J.

Smith, Elroy George (Jun. '08; Assoc. M. '17; M. '31), Cons. Engr., 313 Herald Bldg., Augusta, Ga.

Starkweather, John Burr (Jun. '22; Assoc. M. '31), Vice-Pres., Starkweather & Broadhurst, Inc., 79 Milk St., Boston, Mass.

Tramp, George Dewey (Jun. '25; Assoc. M. '31), County Road Engr., Dickinson County Road Comm. (Res., 521 East F St.), Iron Mountain, Mich.

Weiner, Bernard Louis (Jun. '23; Assoc. M. '31), Asst. Engr. and Structural Designer, Westchester County Park Comm., 72 West Pondfield Rd., Bronxville (Res., 985 Adee Ave., New York), N.Y.

White, Harry Lively, Jr. (Jun. '24; Assoc. M. '31), Asst. Engr., Norfolk & Portsmouth Belt Line R.R., Norfolk, Va.

Resignations

Greenleaf, Donald Leal, Assoc. M., resigned July '31.

Griffin, Wray Wallace, Assoc. M., resigned July '31.

Deaths

Caples, Martin Joseph. Elected Assoc. M., Nov. 4, 1891; M., Oct. 4, 1899; died July 29, 1931.

Coddington, Samuel Cecil. Elected M., Dec. 31, 1913; died June 13, 1931.

Dater, Philip Herrick. Elected Assoc. M., Apr. 5, 1905; M., Mar. 14, 1916; died July 12, 1931.

Keller, Traugott Francis. Elected M., Jan. 16, 1922; died July 10, 1931.

Moxham, Arthur James. Affiliate Nov. 2, 1887; died May 16, 1931.

Phelps, William Collins. Elected M., Oct. 2, 1907; died June 19, 1931.

Volckman, George William. Elected M., Mar. 5, 1912; died July 15, 1931.

TOTAL MEMBERSHIP AS OF AUGUST 7, 1931

Members	5,890
Associate Members	6,347
Corporate Members	12,237
Juniors	2,722
Honorary Members	15
Affiliates	133
Fellows	6
Total	15,113

Men and Positions Available

These items are from information furnished by the Engineering Societies Employment Service with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices, and the fees is to be found on page 97 of the 1931 Year Book of the Society. Unless otherwise noted, replies should be addressed to the key number, Engineering Societies Employment Service, 31 West 39th Street, New York, N.Y.

Men Available

Engineer Executive; M. Am. Soc. C.E.; 55; experienced structural steel, plain and reinforced concrete, and general heavy construction of industrial plants, buildings, electric railways, shipyards, and power plant; 14 years U.S. Steel Corporation. Open for immediate engagement as engineer, architect's engineer, chief draftsman, or construction superintendent, in sales, purchasing, or organization. Central Eastern States preferred. A-4499.

Civil Engineer; M. Am. Soc. C.E.; graduate; licensed two states; over 20 years experience design, construction of industrial plants, hydro-electric developments, and institutional and housing groups for both state and private interests. Supervision of entire work from inception to completion, including correlation of various elements entering into large propositions. B-2835.

Civil Engineer; Assoc. M. Am. Soc. C.E.; 34; married; thorough university training; 9 years experience in design, estimates, power studies, report work, appraisals of hydro-electric developments, irrigation, flood control, and drainage work; 7 years with Stone and Webster Engineering Corporation. Fully competent to plan and direct work. Location immaterial. B-7050.

Junior Engineer; 25; recent graduate; unmarried; familiar with Eastern and Western States; 3 years experience in building construction; also surveying and drafting. Desirous of obtaining experience in civil engineering. Willing to travel anywhere. C-9565.

Civil Engineer; 23; graduate; specialized in hydraulics, class 1931. Ambitious and resourceful. Looking for opening with reputable and enterprising concern. Ready to go any place where there is opportunity for advancement. Can furnish excellent references. C-9566.

Civil Engineer; M. Am. Soc. C.E.; 1 year Reclamation Service; 13 years Panama Canal, dredging, municipal departments, and in charge of construction of pump-stations, reinforced concrete dock, and filtration plant; municipal work in Southern district including Panama; 12 years in Peru, on water supply, municipal work, and general building construction. Speaks Spanish. Position General Superintendent or Manager. C-9448.

Civil Engineer; M. Am. Soc. C.E.; technical graduate; Arkansas license; 20 years varied experience, drainage, flood control and levees, highway surveys, pavements, sewers, water works, and highway bridges. Extensively experienced in earth-moving projects, supervising projects, also designing and compiling maps and plans. Available immediately. C-1106.

Engineering Executive; M. Am. Soc. C.E.; 25 years engineering, executive, and construction experience on municipal, highway, and allied lines. Responsible position desired. Middle States preferred. Available on short notice. C-8700.

Assistant Chief Engineer and Office Engineer; Assoc. M. Am. Soc. C.E.; 29; married; 2 years college; 10 years experience—5 in tropics; reports, irrigation, drainage design, construction, and geodetic and plane table work. Knowledge Spanish. Desires connection office or field organization. Capable taking complete charge. Short notice. Western United States preferred. C-9630.

Graduate Engineer; Jun. Am. Soc. C.E.; 26; 5 years experience; competent as instrumentman, inspector, resident engineer, draftsman, and computer on highway work; experienced in design of roadway plans. Wishes employment at once, anywhere in United States. Can furnish references and letters of recommendation. C-4601.

Graduate Civil Engineer; 24; 1931 graduate; unmarried; desires to make connection with opportunity to gain experience and advancement. Best references. Available immediately. C-9611.

Civil Engineer; Jun. Am. Soc. C.E.; 23; recent graduate; desires position in connection with hydraulic or sanitary work, office or field; 3 months transitman on construction; 3 months draftsman on pipe-line survey; 2 months draftsman on piping layout and design. Location immaterial. C-9626.

Engineering and Sales Executive; M. Am. Soc. C.E.; 30 years varied engineering experience, domestic and foreign; last 15 years, U.S. district manager, handling sales negotiations for internationally known engineering and contracting firm. Wide contacts; excellent references. Willing to arrange personal conference with responsible representative in New York City. C-9631.

Civil Engineer; M. Am. Soc. C.E.; technical graduate, with broad experience and executive ability; will consider position requiring tact, loyalty, energy, and resourcefulness. Experience includes teaching, water filtration, irrigation, road, railway, and land surveys, construction by force account and contract, sales, sales promotion, and trade association work. B-3340.

Hydraulic and Structural Engineer; Assoc. M. Am. Soc. C.E.; 34; graduate, Worcester Polytechnic Institute; expert on design of hydraulic structures, reinforced concrete, economic studies, power development, heavy foundations, indeterminate structures, and strain energy research. Responsible experience on Conowingo, Osage, and Rock Island hydro-electric projects; Cahokia and Long Beach steam plants; and Lackawanna electrification. B-4078.

Civil Engineer; Jun. Am. Soc. C.E.; 28; married; graduate, Massachusetts Institute of Technology; 7 years active experience in field and office—excavations, railroad, subway, and steel construction, hydro-electric developments, design of hydraulic and reinforced concrete structures, sewers, dams, power house, and foundations. Available immediately. Location immaterial. Excellent references. C-5735.

Civil Engineer; recent graduate, Rensselaer Polytechnic Institute; single; healthy. No practical experience. Desires position with any reliable engineering company, state highway department, or as instructor in a college. No preference as to location. Available immediately. Reasonable salary. C-9625.

Engineering Executive; M. Am. Soc. C.E.; technical graduate; 17 years experience engineering, executive, highway construction, municipal, sanitary, drainage, railroads, and allied lines in United States, Latin America, and Europe. Can handle projects from preliminary investigations to completion. Good personality. Desires position in major or minor executive capacity. Location immaterial. C-9650.

Civil Engineer; 30; recent college graduate; 3 years experience on large lumber and logging job; 5 years experience in building construction; desires position in designing, drafting, surveying, or construction. Will go anywhere. C-9614.

Civil Engineer; Assoc. M. Am. Soc. C.E.; 38; married; graduate, German University; 15 years experience. Responsible position desired—preferably accurate calculations on design and estimate for steel and concrete. C-9616.

Graduate Engineer; M. Am. Soc. C.E.; married; New York license; 26 years experience engineering design, waterway construction, river dams, locks, docks, piers, port equipment, municipal water supply, sewers, bridges, state highways, and parks; 1 year on French port. Desires position capitalizing tact, energy, and personality with construction department for contractor, consultant, or municipality. A-3314.

Civil Engineer; M. Am. Soc. C.E.; A. R. E. A.; 25 years general experience on important railway and bridge construction, highways, irrigation, water supply, drainage, valuations, reconnaissances, and investigations for various large projects, including United States and foreign. Fluent knowledge of French including technical engineering terms. Knowledge of Spanish. B-1476.

Civil Engineer; Assoc. M. Am. Soc. C.E.; 44; married; graduate civil engineer; 20 years experience—9 as instructor and assistant professor, 1 on construction, and 7 as principal assistant on design of hydro-electric developments. Desires to teach mechanics, and structural and

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hydraulic engineering; in order of preference. Available for year 1931–1932. B-4813.

JUNIOR CIVIL ENGINEER; $5^1/_2$ years at University of Pittsburgh; majored in chemistry first three years; graduated June 1931; degree of B.S. in C.E.; Student Member of the Society. C-9658.

GRADUATE CIVIL ENGINEER; Jun. Am. Soc. C.E.; Jun. Am. Soc. Testing Materials; Soc. Am. Military Engineers; 24; single; 6 months municipal engineering; 2 years as structural research engineer with large company. Can furnish references. Available immediately. Pennsylvania or New Jersey preferred, but will go anywhere. C-5768.

EXECUTIVE—ENGINEER AND CONTRACTOR; M. Am. Soc. C.E.; heavy construction, bridges and buildings, steel, masonry, concrete, and timber. Creditable achievements in project promotion and engineering contract-sales service; personal contact activities highly efficacious; firmly established influential friendships. No liabilities. Moderate capital. Unexcelled references. Seeks executive engagement or financial partner. C-9265.

GRADUATE CIVIL ENGINEER; Jun. Am. Soc. C.E.; 30; married; B.S. and M.S. degrees in C.E.; advanced study design, control of concrete mixtures; 6 months assistant city engineer; $4^1/_2$ years railroad construction and maintenance. Location immaterial. Available for engineering construction, maintenance, or as college instructor. C-9537.

ENGINEERING ADVISER ON GOLD DOLLAR INVESTMENTS; M. Am. Soc. C.E.; for developments in China; just now very attractive; years of experience there; ready to assist. Knows the exchange game well. C-2148.

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 8 years experience, including buildings, railroads, subway construction, steel, concrete design, drafting, checking, and estimating; also field engineering. Connection with construction company, consultant, contractor, or architect desired; capacity to assume responsibility and interest to promote productive activity; preferably field engineering, construction, or related work. C-2605.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 32; married; graduate; 3 years experience in highway design and construction; 7 years experience in design and construction of public works improvements, including pavements, sewers, water supply, and sewage treatment. Available September 1. Location immaterial. C-9673.

HYDROGRAPHIC INVESTIGATOR; Jun. Am. Soc. C.E.; B.S. and M.S. in C.E., Massachusetts Institute of Technology; post graduate courses, municipal, sanitary, and hydro-electric engineering. Experience in hydro-electric investigations. During two years developed and directed hydrographic service for large public utility in South America. Location immaterial. C-3062.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; graduate, University of Kentucky, 1929; varied experience in highway location and construction in South and Middle West; railroad grade elimination projects in East and academic work. Desires connection as instructor, or highway, railroad, or construction work, with opportunity to advance. Will go anywhere, but New York State preferred. C-9606.

GRADUATE CIVIL ENGINEER; 22; single; graduate, Newark College of Engineering; 2 years experience as inspector in structural steel plant; 6 months experience clerical work in civil engineering office. Desires position with construction or engineering firm. Prefers vicinity of New York City, but will go anywhere. Excellent references. Available immediately. C-9592.

STRUCTURAL ENGINEER; M. Am. Soc. C.E.; 36; married; C.E. graduate; 14 years broad experience steel, concrete, timber with public utility engineers, railroad, consulting engineer, including responsibility for structural design, estimates, valuations, reports; 8 years teaching structural subjects in night school and college. Desires position in private practice or professorship. B-9683.

EXECUTIVE ENGINEER; Assoc. M. Am. Soc. C.E.; licensed New York State engineer; 10 years with engineers and contractors on concrete and industrial buildings; 10 years as chief engineer for several large textile mills. Experienced in designing, detailing, specifications, supervision of buildings, including plumbing, heating, fire protection, maintenance, process developments, and industrial engineering research. Location immaterial. C-494.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; Cornell graduate; 12 years varied experience, design, construction, and executive duties, hydro-electric plants, manufacturing plants, and institutional buildings. Desires responsible position with established firm of consulting engineers, architects, or contractors engaged in designing and building. Prefer Southern or Eastern location. Available immediately. C-9622.

EXECUTIVE, CONSTRUCTION ENGINEER, SUPERINTENDENT; M. Am. Soc. C.E.; 37; married; 15 years on large hydro-electric projects, concrete, earth, and rock-fill dams, and all kinds of excavation; charge of construction, cost estimates, and contract bids. Experience includes power estimates and designs. Employed by large utility and contracting companies. United States and foreign locations, including South America. C-9689.

CIVIL ENGINEER; application has been made for Junior membership in the Society; 25; single; 1931 graduate of cooperative plan engineering school; approximately $1^1/_2$ years experience, instrumentman, surveying; 2 months, instrumentman, reinforced concrete construction. Available on short notice. Location and salary open. C-9690.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 40; New Jersey registration; married; 18 years experience as engineer, estimator, superintendent, and project manager on buildings of all types and concrete structures, such as subways, state roads, and bridges. Thoroughly qualified to take responsible charge. Excellent knowledge of costs; modern methods of construction. C-4211.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 30; graduate, 1924; 7 years thorough experience all phases of design and construction, reinforced concrete and steel highway bridges, United States and Central America; able to plan and direct work. Working knowledge of Spanish. Available immediately; location immaterial; excellent references. C-9343.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; A.S.M.E.; 39; technical graduate; native of Argentine; 20 years experience in America; exceptionally well familiarized with conditions in South America; offers his services as representative, travel or sales engineer, or in making technical reports and investigations. Can report at once. C-1165.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 30; married; graduate; professional engineering license, New York; 8 years experience as structural designer, assistant to consulting engineer; steel, concrete designing, investigations, reports, supervision of construction all classes of buildings and foundations; desires position with architect, consulting engineer, or contractor in New York. B-7382.

RECENT BOOKS

New books of interest to Civil Engineers, recently donated by the publishers to the Engineering Societies Library, will be found listed here. A comprehensive statement regarding the service which the Library makes available to members is to be found on pages 87 to 89 of the Year Book for 1931. The statements made regarding the books are taken from the books themselves and this Society is not responsible for them.

BUILDING HEIGHT—BULK AND FORM. By G. B. Ford, A.B. Randall, and L. Cox. vol. 2. Harvard City-Planning Studies, Cambridge, Harvard University Press, 1931. 188 pp., illus., diagrs., charts, tables, 10 × 7 in., cloth. $3.50.

This investigation was undertaken primarily to determine the optimum size of buildings on high-priced land, and to suggest ways in which zoning may be used to facilitate desirable conditions. The results of the study will be of interest to engineers, legislators, and others interested in the erection of commercial and apartment buildings.

L'EMPLOI DES UNITÉS DANS LA PRATIQUE DES CALCULS. By F. Bétrancourt. Paris, Dunod, 1931. 91 pp., tables, 9 × 6 in., paper. 16 fr.

A collection of the geometrical, mechanical, electrical, magnetic, and optical units of measurements, with definitions and formulas, intended as an aid to calculators.

INDUSTRIAL ORGANIZATION. By H. Rubey. Boston and New York, Ginn & Co., 1931. 308 pp., diagrs., charts, tables, maps, 9 × 6 in., cloth. $2.80.

A brief, but comprehensive course covering such fundamentals of business as promotion, finance, management, organization, marketing, personnel, accounting, estimating, and valuation. The book is intended primarily to give the engineer some knowledge of business, as a preparation for executive and managerial work. It will also be useful to others who wish a general survey of this field.

INTERNATIONAL CRITICAL TABLES OF NUMERICAL DATA, PHYSICS, CHEMISTRY, AND TECHNOLOGY. By the National Research Council vol. 7. New York, McGraw-Hill Book Co., 1930. 507 pp., 11 × 9 in., cloth. $84.

This is the final volume of this work, in which the best values for an immense number of constants and numerical data of physics, chemistry, and engineering are recorded for the convenience of research workers. The values are based upon all the recorded work of the world previous to 1924, as recorded in the "Annual Tables of Constants and Numerical Data." The values given in the present work have been chosen by competent authorities as the most satisfactory. References to the original publications are included. The work is indispensable in research laboratories and libraries of science.

NEIGHBORHOODS OF SMALL HOMES: ECONOMIC DENSITY OF LOW-COST HOUSING IN AMERICA AND ENGLAND. By R. Whitten and T. Adams. vol. 3. Harvard City-Planning Studies, Cambridge, Harvard University Press, 1931. 205 pp., illus., diagr., charts, maps, tables, 10 × 7 in., cloth. $3.50.

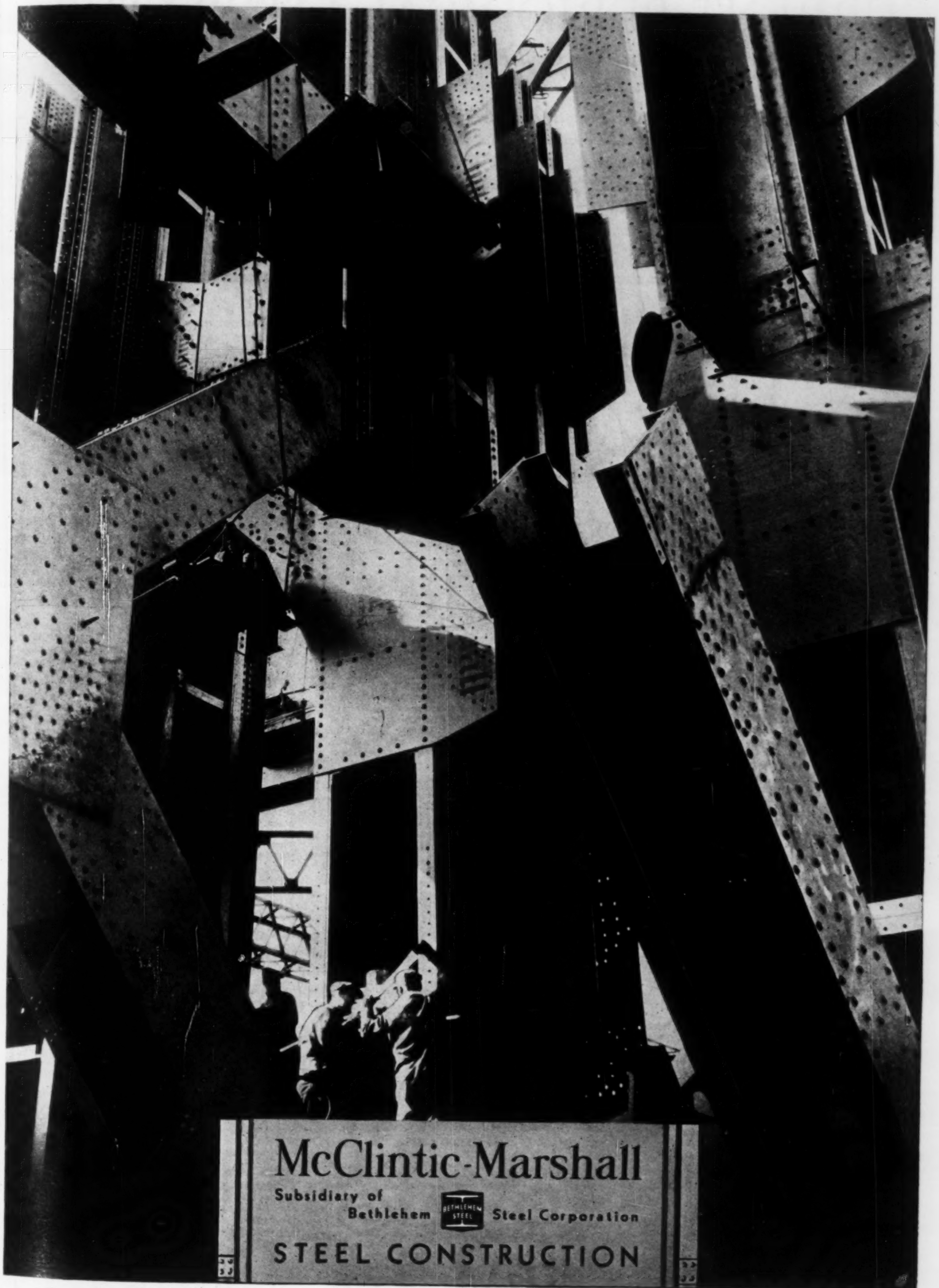
As a contribution to the knowledge of certain closely interrelated problems of city planning and housing, this volume embodies the results of an investigation into less intensive uses on low-cost land. The study was to determine the cost of complete city improvements and adequate housing; the effects of lot size, open space, and block and street layout on house cost; the best methods of recognizing economic and social considerations in zoning; and the control of land subdivision conditions in America and England.

SAGS AND TENSIONS IN OVERHEAD LINES. By C. G. Watson. London and New York, Isaac Pitman & Sons, 1931. 192 pp., diagrs., charts, tables, 9 × 6 in., cloth. $3.75.

The author presents simple methods for determining sags and tensions which are accurate, yet require no further mathematical knowledge than an ability to plot and read graphs, and perform simple arithmetical operations. All the cases ordinarily met in practice are discussed and explained. Tables and graphs of hyperbolic and catenarian functions and a table of tangents are given.

TABELLENBUCH FÜR DIE BERECHNUNG VON KANÄLEN UND LEITUNGEN. By E. Wild and O. Schöberlein. Berlin, Julius Springer, 1931. 57 pp., diagrs., charts, tables, 10 × 7 in., cloth. 25.50 r.m.

These tables give numerical values for the discharge, velocity, hydraulic radius, and hydraulic gradient of conduits of all the cross sections in common use for sewers, water mains, and drainage. The tables, based on Kutter's formula, are in metric units.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Photograph shows detail of portion of New Jersey Tower, George Washington Bridge across Hudson River at New York City. Steel fabricated and erected by McClintic-Marshall.

CURRENT PERIODICAL LITERATURE

Abstracts of Articles on Civil Engineering Subjects from Magazines in This Country and in Foreign Lands

Selected items from the current Civil Engineering Group of the Engineering Index Service, 29 West 39th Street, New York, N.Y. Every article indexed is on file in The Engineering Societies Library, one of the leading technical libraries of the world. Some 2,000 technical publications from 40 countries in 20 languages are received by the Library and are read, abstracted, and indexed by trained engineers. With the information given in the items which follow, you may obtain the article from your own files, from your local library, or direct from the publisher. Photoprints will be supplied by this library at the cost of reproduction, 25 cents per page, or technical translations of the complete text may be obtained when necessary at cost.

BRIDGES

Clearances. Text of War Department's Refusal of Application for New Hudson River Bridge. *Eng. News-Rec.*, vol. 106, no. 26, June 25, 1931, p. 1067, 1 fig. Original text and editorial comments; mast and funnel heights of vessels entering New York Harbor.

Concrete Arch. Reinforced Concrete Bridges, T. J. Gueritte. *Engineer*, vol. 151, no. 3935, June 12, 1931, pp. 662–663. Cost of temporary work for important spans, which has often been unduly high because sufficient attention was not paid to design, is one cause of comparatively slow increase in span of concrete bridges; this question received careful consideration in the case of the concrete viaduct at Elorn-Plougastel; weight of reinforced-concrete arch of great span would be hardly half that of equivalent steel arch, and would cost considerably less. Before Inst. Structural Engrs. and Brit. Section of Société des Ingénieurs Civils de France.

Design. Modern Bridges, O. M. Ayrton. *Roy. Inst. Brit. Architects—Journal*, vol. 38, no. 14, May 16, 1931, pp. 479–487 and (discussion) 488–495, 15 figs. Review of bridge building practice since Eighteenth Century, with special reference to bridges in Great Britain and France.

Reinforced Concrete Bridges, A. L. L. Baker. *Concrete and Constructional Eng.*, vol. 26, no. 6, June 1931, pp. 343–352, 14 figs. Calculations for design of beams and slabs of bridge floors.

Floors. Bridge Floor Built of Pre-Cast Concrete Slabs Secured by Welding. *Eng. News-Rec.*, vol. 106, no. 22, May 28, 1931, p. 894, 2 figs. Description of floor of new Wabash Avenue, 232-ft., double-leaf, bascule bridge, over Chicago River, made of pre-cast slabs of reinforced concrete, and having steel lugs which are welded to steel-deck framing; details of concrete floor slabs.

Kill van Kull. The Kill van Kull Bridge, New York, O. H. Ammann. *Engineer*, vol. 151, nos. 3935 and 3936, June 12, 1931, pp. 647–650 and 654, and June 19, pp. 670–672, and 682, 23 figs. Information obtained from two Progress Reports on structure, second of which is dated Mar. 1931; both are signed by Chief Engineer of Bridges to Port of New York Authority. Descriptions of bridge have been previously indexed from various sources. See Engineering Index 1930, p. 236.

Lift, Seattle. Fourteenth Avenue South Bridge, Seattle, T. D. Hunt. *West. Construction News*, vol. 6, no. 11, June 10, 1931, pp. 287–288, 2 figs. Bridge consists of Scherzer rolling lift double-leaf span with clear channel opening between fenders of 125 ft., flanked on each end by two steel-deck spans 86 ft. 6 in. and 79 ft. 2 in. center of bearings; 240 ft. of 38-ft. span flat-slab concrete approach, and 120 ft. of concrete retaining wall.

Reconstruction. Reconstruction of Approach Spans, Royal Albert Bridge, Saltash, F. Gibbons. *Inst. Civil Engrs.—Min. of Proc.*, vol. 230, for mtg. Feb. 25, 1931, pp. 115–124, 1 plate at end of book. Indexed in Engineering Index 1930, p. 238, from *Min. of Proc.*, no. 4776, 1930.

Reconstruction of Liskeard Viaduct; and Scheme for Reconstruction of the Approach Spans of the Royal Albert Bridge, Saltash, H. D. Smith. *Inst. Civil Engrs.—Min. of Proc.*, vol. 230, for mtg. Feb. 25, 1930, pp. 100–114, 1 plate at end of book. Indexed in Engineering Index 1930, p. 234, from *Min. of Proc.*, no. 4760, 1930.

Steel Arch, Great Britain. Tyne Bridge, Newcastle, D. Anderson. *Inst. Civil Engrs.—Min. of Proc.*, vol. 230, for mtg. Feb. 25, 1930, pp. 167–168 and (discussion) 189–202, 2 plates at end of book. Indexed in Engineering Index 1930, p. 236, from *Min. of Proc.*, no. 4771, 1930.

Steel Truss, Design. K-Type Trusses Selected for 225-Ft. Highway Bridge, P. Andersen. *Eng. News-Rec.*, vol. 106, no. 24, June 11, 1931, p. 982, 2 figs. In designing two 225-ft. trusses for highway bridge, over Sulphur River in Miller County, Ark., K-arrangement of web members was considered preferable to more commonly used Baltimore truss, which would require secondary members to reduce slenderness ratio of top chord and verticals.

Suisun Bay, Calif. The Martinez-Benicia Bridge, Suisun Bay. *Engineer*, vol. 151, no. 3932, May 22, 1931, pp. 562–563, 5 figs., partly on p. 574. This 5,603-ft. bridge is the longest and heaviest two-track structure west of the Mississippi River; unusual features in construction; steel superstructure consists of seven 526-ft. through Warren truss spans; two deck spans of 264 and 504 ft. length, one 326-ft. vertical-lift-span; 560-ft. viaduct at south end; and 220-ft. viaduct at north end. See references to previous descriptions in Engineering Index 1930, p. 238.

Suspension, Golden Gate. The Golden Gate Bridge. *Eng. News-Rec.*, vol. 106, no. 22, May 28, 1931, pp. 890–893, 4 figs.; see editorial comment, p. 877. Structural details of suspension bridge with main span 4,200 ft. long, incorporating definite provisions for resisting earthquake forces; anchorages; cables and suspenders; tower and floor design; stiffening trusses; loads, stresses, and materials; construction program.

Sydney, Australia. Silicon-Steel Arch, 1,650 Ft. in Length, Spans Harbor at Sydney, Australia. *Eng. and Contracting*, vol. 70, no. 6, June 1931, pp. 133–136, 8 figs. Description of bridge previously indexed from various sources. See also Engineering Index 1930, pp. 236–237.

Vibrations. The Locomotive and Bridge-Oscillation, W. E. Dalby. *Inst. Civil Engrs.—Lecture*, at mtg. Dec. 10, 1929, 1930, 13 pp., 1 fig. Action of locomotive on roadbed; experimental demonstration, by means of models, of action on track of both forces and couples of balanced and unbalanced engines; motion in straight line at varying speed; synchronism; comparison of action of two-cylinder and four-cylinder locomotives on bridge.

Widening. Scotswood Suspension Bridge, Newcastle-Upon-Tyne. *Surveyor*, vol. 79, no. 2051, May 15, 1931, p. 542, 2 figs. Strengthening and widening of suspension bridge having total length of 910 ft., distance between centers of river piers being 368 ft. with two side spans of 131 ft. each; widening bridge carriageway from 16 ft. 10 in. to 20 ft.

BUILDINGS

Auditoriums, Heating and Ventilation. Heating and Ventilating Auditoriums and Theaters, C. L. Hubbard. *Plumbers Trade Journal*, vol. 91, no. 1, July 1, 1931, pp. 12–13, 4 figs. Methods of conditioning air in assembly halls and amusement buildings with and without balconies; remodeling tips for progressive contractors; typical heating and ventilating system for motion picture theater; varying outside recirculated air; schemes for better air distribution; heat required for warming air.

Concrete. Thesis on the Study of Esthetic Design of Reinforced Concrete on the Continent, H. B. Rowe. *Structural Engr.*, vol. 9, no. 6, June 1931, pp. 206–216, 26 figs. Notes on modern architectural trends in Belgium, Holland, and Germany. (To be continued.)

Condition Surveying. Rules for Condition Surveys of Buildings, W. T. McIntosh. *Eng. News-Rec.*, vol. 107, no. 1, July 2, 1931, p. 26. Discussion by H. deB. Parsons, of paper previously indexed from issue of June 18, 1931.

Electric Wiring. Club House Electrical System, A. P. Seeley. *Elec. World*, vol. 97, no. 20, May 16, 1931, pp. 900–903, 5 figs. Interlocking of lights and ventilation systems; multiplicity of outlets for electric service, radio signal, and communication systems; facilities for future service demands, reliability, and safety.

Gas Heating. The Use of Gas for Heating Industrial Structures, F. M. Reiter. *Gas Age-Rec.*, vol. 67, no. 15, Apr. 11, 1931, pp. 537–540, and 544. Differences in problem of sale of gas for home heating and industrial heating; advantages of gas yield in heaters, contrasted with centralized steam heating unit; competition with coal; cost of gas heating. Before East. Natural Gas Regional Sales Conference.

Heating and Ventilation. Large Space Warm Air Heating, O. W. Kothe. *Heat., Piping, and Air Conditioning*, vol. 3, no. 6, June 1931, pp. 479–482, 6 figs. Typical heating and ventilation systems installed in large spaces as, factories, open buildings, auditoriums, and assembly halls.

High Building, Construction. Tying a New Addition to a Leaning Building, W. C. Spiker. *Eng. News-Rec.*, vol. 106, no. 25, June 18, 1931, pp. 1004–1005, 3 figs. Connection of new 7 to 17-story addition to First National Bank 17-story building, in Atlanta, Ga., aggravated by lean of old building due to settlement; suggested design for continuous welded girders.

Moving. Business as Usual While Moving an Eight-Story Steel-Frame Building. *Eng. News-Rec.*, vol. 107, no. 1, July 2, 1931, pp. 4–8, 7 figs. Indianapolis telephone building shifted 52 ft. sideways and turned 90 deg.; special steel framework transmits column loads to rollers running on grillage of closely spaced rails; principal moving force is from 18 manually operated ratchet jacks.

Silos, Reinforced Brick. A Practical Test for Reinforced Brickwork. *Brick and Clay Rec*, vol. 78, no. 10, May 19, 1931, pp. 532–534, 5 figs. At Wedron, Ill., near Ottawa, the Wedron Silica Company, is building two sand storage bins or silos—one 25 ft. 1 in. in diam. and the other 16 ft. $2^{1}/_{2}$ in.; both will rise to height of 52 ft. above grade line; construction of these bins represents first major example of use of reinforced brickwork in this part of world; advantages of reinforced brickwork.

Underpinning. Heavy Building Underpinning Nassau-Broad St. Subway, W. T. McIntosh. *Eng. News-Rec.*, vol. 107, no. 1, July 2, 1931, pp. 10–12, 4 figs. Success of open-cut method of construction through quicksand area of lower Manhattan depended upon providing proper support for adjacent buildings; 4,800-ft. double-track route cost more than $10,000,000; new method with 36-in. caissons; pipe pile underpinning National Bank of Commerce Building; pit underpinning.

Wind Bracing. Expert Testimony on Wind Design for Tall Buildings—III, Chord Deflections Control Web System Design, H. V. Spurr. *Eng. News-Rec.*, vol. 106, no. 25, June 18, 1931, pp. 1012–1014, 2 figs. Model test data are of limited value; visualizing wind-loading effects; typical wind connections in 900-ft. Manhattan Company Building, New York; controlling rigidity; cantilever methods for tall slender towers; columns dominate wind design in super-tall building.

Expert Testimony on Wind Design for Tall Buildings—V, Some Problems of the Future. *Eng. News-Rec.*, vol. 107, no. 1, July 2, 1931, pp. 20–22. A Veteran's Views, R. Fleming; Pressure Variation with Height, F. A. Randall; Study Behavior of Present Buildings, H. G. Balcom; Rigidity, the Modern Problem, A. N. Van Vleck; Wind Design an Art, J. W. Pickworth; Lateral Resistance in Walls, H. D. Dewell.

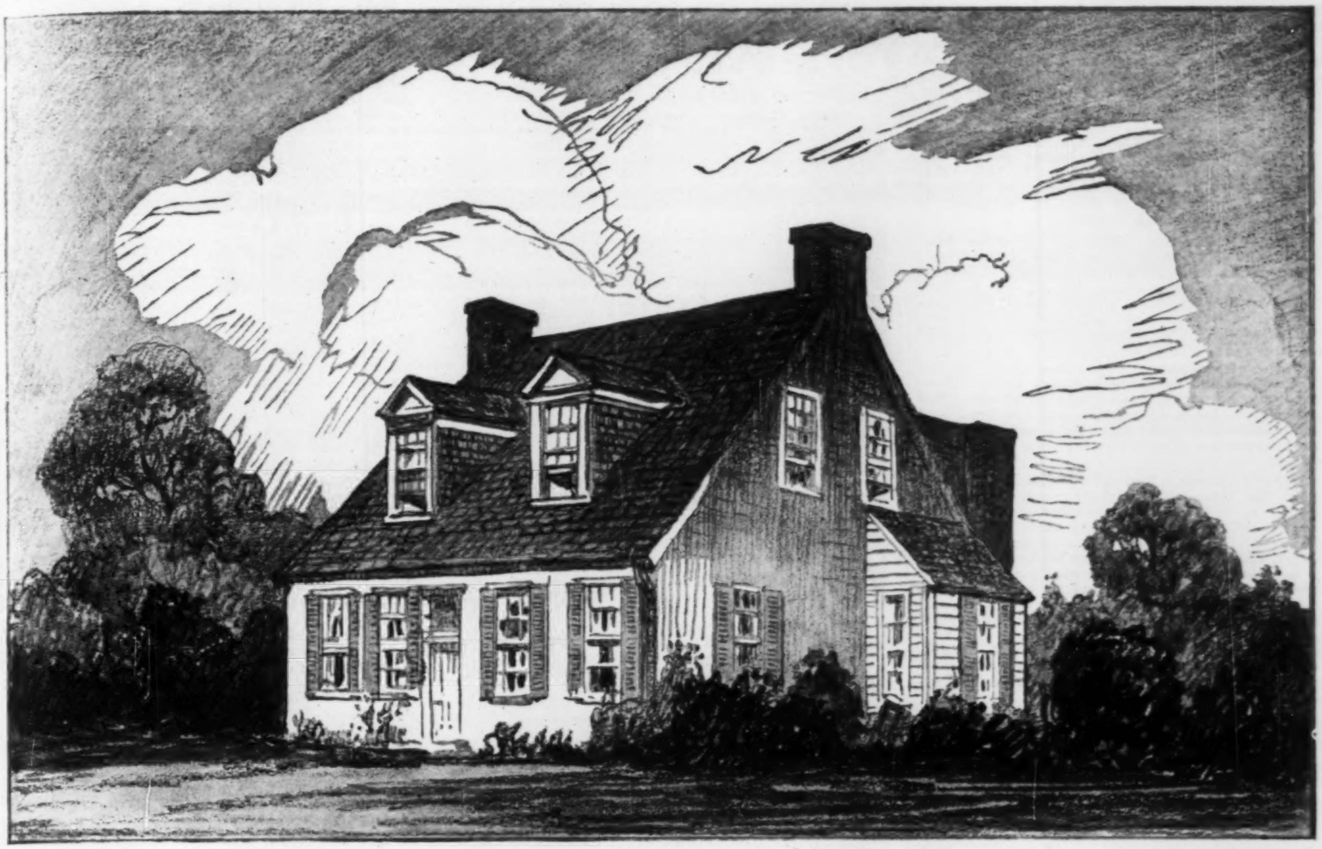
Wind Design of Tall Buildings—II—Assumed Loads and Fiber Stresses, A. Smith. *Eng. News-Rec.*, vol. 106, no. 24, June 11, 1931, pp. 971–974, 4 figs. Unit fiber stresses; comparative column formulas together with strengths and yield points of tested columns; unit wind loads; proportioning frame.

CITY AND REGIONAL PLANNING

Boston. An Outline of the Boston Thoroughfare Plan, R. Whitten. *Boston Soc. Civil Engrs.—Journal*, vol. 18, no. 5, May 1931, pp. 162–174 and (discussion) 174–175, 8 figs. Brief discussion of nine major projects of plan.

CONCRETE

Construction. Europe's Contribution to the Ferro-Concrete Style, F. S. Onderdonk. *Engrs. and Eng.*, vol. 48, no. 6, June 1931, pp. 127–133,



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9 figs. Examples of new style in concrete architecture.

Construction, Slopes. The Technic of Placing Concrete on Steep Slopes without Forms, I. E. Burks. *Eng. Journal*, vol. 14, no. 6, June 1931, p. 350. Discussion of paper before Eng. Inst. Can. previously indexed from issue of Mar. 1931.

Disintegration. The Action of Sulfate Water on Concrete, D. G. Miller and P. W. Manson. *Pub. Roads*, vol. 12, no. 3, May 1931, pp. 64–87, 12 figs. Detailed results of tests of 2 by 4-in. specimens of concrete immersed in Medicine Lake, South Dakota.

Early Strength. Effect of Heating Materials and Appliances on the Rate of Hardening of Rapid-Hardening Portland Cement, N. Davey. *Concrete and Constr. Eng.*, vol. 26, no. 6, June 1931, p. 359–364, 1 fig. Test specimens prepared with materials and appliances at 15.5 deg. cent. and at 70 deg. cent. at time of mixing and placing.

Fire Resistance. Fire-Resistive Values of Concrete and Concrete Masonry Units. *Concrete*, vol. 38, no. 6, June 1931, pp. 23–24. Requirements for walls, partitions, columns, and other structural parts given in a new report by Department of Commerce Building Code Committee.

Manufacture. The Proportioning of Concrete for Strength, Durability, and Impermeability, A. T. Goldbeck. *Crushed Stone Journal*, vol. 7, no. 5, May 1931, pp. 5–12 and 25–26, 10 figs. Principles of manufacture of ready-mixed concrete having certain desired properties—such as given crushing strength or transverse strength, without being limited as to proportions; effect of water–cement ratio; characteristics of cement; gradation of aggregate; surface coatings; method of curing; design of concrete for given compressive strength; proper water–cement ratio to use for different classes of structures. Before Nat. Ready-Mixed Concrete Assn.

Mixers, Testing. A Comparison of Continuous with Batch Mixers in Plant Operation, B. Wilk. *Am. Concrete Inst.—Journal*, vol. 2, no. 10, Feb. 1931, pp. 1305–1306. Discussion by H. B. Emerson and A. G. Conrow, of paper previously indexed from issue of Feb. 1931.

Mixing, Admixture, Hydraulic Lime. Hydraulic Lime in Concrete, G. W. Hutchinson. *Eng. News-Rec.*, vol. 106, no. 24, June 11, 1931, pp. 974–976, 4 figs. Effect of combinations of hydraulic lime and portland cement on concrete strength, of hydraulic lime on volume of concrete, and on concrete strength when hydraulic lime is added to obtain given paste content; change in effect of water ratio on concrete strength with various combinations of hydraulic lime and portland cement.

Plants. Fire-Proof Structures Replace Original Plant Destroyed by Flames, M. F. Beisber. *Pit and Quarry*, vol. 22, no. 6, June 17, 1931, pp. 36–43, 22 figs. New crushing and ready-mixed concrete plants of Birmingham Slag Company, Birmingham, Ala.

Ready-Mixed. Control Factors in Ready-Mix, F. C. Wilcox. *Concrete Products*, vol. 40, no. 6, June 1931, p. 41. Wet plant and dry plant systems of making and delivering ready-mixed concrete from standpoint of control.

Design and Operation of Central Mixing Plants, F. I. Ginsberg. *Am. Concrete Inst.—Journal*, vol. 2, no. 10, June 1931, pp. 1237–1249, 2 figs. Progress Report of Committee 603; types of central plants; plant equipment and methods; handling cement in bulk; central mixing plants; agitator or non-agitating truck delivery; central proportioning plants; truck mixer delivery; combination plant; delivery of concrete; miscellaneous problems.

Marketing Aggregates as Ready-Mixed Concrete. *Rock Products*, vol. 34, no. 11, May 23, 1931, pp. 98–101, 16 figs. Practice of Big Rock Stone and Material Co., Little Rock, Ark., which produces both stone and sand, furnishing these either as aggregates or as concrete.

Producing Aggregates and Concrete for the Beauharnois Power Project. *Pit and Quarry*, vol. 22, no. 4, May 20, 1931, pp. 36–40, and 44, 13 figs. Crushed-stone screening and concrete mixing accomplished in single building.

Specifications. Report of Committee C-9 on Concrete and Concrete Aggregates. *Am. Soc. Testing Materials—Advance Paper*, no. 51, for mtg. June 22–26, 1931, 118 pp., 22 figs. (4 cards—card 3.) Tentative specifications for curing portland-cement concrete slabs by surface application of calcium chloride, and wet coverings; method of test for structural strength of fine aggregate using constant water–cement ratio mortar, and for apparent specific gravity of coarse aggregates in saturated condition; method of routine analysis of cement content of portland-cement concrete; proposed revised tentative specifications for concrete aggregates.

Supports, Coal Mines, and Mining. Notes on the Use of Reinforced Concrete Underground, A. Marshall and J. Chadwick. *Inst. Min. Engrs.—Trans.*, vol. 81, pt. 2, May 1931, pp. 190–200 and (discussion) 200–201. Previously indexed from Colliery Guardian, Apr. 17, 1931.

Time Study. Time Study of a Concrete Mixer A. E. King. *Roads and Road Construction*, vol. 9, no. 101, May 1, 1931, pp. 149–151. Average, maximum, and minimum time in seconds, for long series of mixing operators.

CONSTRUCTION INDUSTRY

Costs. Current Construction Unit Prices. *Eng. News-Rec.*, vol. 107, no. 22, May 28, 1931, pp. 912–913, 3 figs. Unit prices bid and description of following: covered concrete reservoir at Springfield, Mass., 288 by 224 ft. with capacity for 11,000,000 gal.; Montrose Harbor retaining wall, Chicago, 4,600 ft. long and 22 ft. high by 39 ft. wide; steel highway bridge and concrete overhead, Wisconsin, consisting of six continuous concrete spans and three deck-truss spans having total length of 531 ft. 5 in.

Current Construction Unit Prices. *Eng. News-Rec.*, vol. 107, no. 25, June 18, 1931, pp. 1032–1033, 4 figs. Unit prices bid and description of following: levee work on Mississippi tributaries totaling 4,913,000 cu. yd.; eight spans of 3-mile viaduct on Jersey Meadows, totaling about 2,500 ft., laying cast-iron water mains at Pembroke, Mass.; Coldbrook-Swift Tunnel near Boston (low bid was $4,978,032).

Current Construction Unit Prices. *Eng. News-Rec.*, vol. 107, no. 1, July 2, 1931, pp. 34–35, 3 figs. Unit prices bid and description of following: bascule bridge over Chicago River (structure is double-leaf trunnion bascule bridge 264 ft. long between center line of trunnions, 108 ft. wide with two 38-ft. roadways separated by 4-ft. center island); lowest bid was $1,591,327; wharf extension at Richmond, Va., to cost $68,910.

Stabilization. Thoughts on Stabilizing Construction. *Eng. News-Rec.*, vol. 106, no. 22, May 28, 1931, pp. 898–899. Symposium consisting of two articles: Stabilization of Employment, R. C. Marshall; Rehabilitating Home Building, F. L. Cranford; direct Federal action limited; timed construction is remedy; definite administration necessary; industry should assume control; conditions that retard home building; revision of building trade wages.

DAMS

Concrete, Construction. Design and Control of Concrete for Diablo Dam, H. F. Faulkner and R. R. Hubbard. *Am. Concrete Inst.—Journal*, vol. 2, no. 10, June 1931, pp. 1307–1310, 3 figs. Discussion by F. R. McMillan, of paper previously indexed from issue of Feb. 1931.

Concrete Gravity, Quebec. Replacement of Concrete Dam on Au Sable River, Caffiaux and Villeneuve. *Contract Rec.*, vol. 45, no. 23, June 10, 1931, pp. 699–701, 4 figs. Structure built in 1908 had disintegrated and required rebuilding; construction work carried out in winter time; new concrete dam of gravity type is 14 ft. wide at top, elevation 102.0 and 100.0, and 44 ft. wide at elevation 60.0; width at base varies according to elevation, from 50 to 56 ft.

Concrete, India. The Lloyd Barrage. *Civil Eng. (Lond.)*, vol. 25, no. 12, May 1931, pp. 617–619, 9 figs. Abstracts of sections from Volumes II and V of report on Sukkur Barrage Project, dealing with results of model experiments carried out with various designs of piers and sills; experiments to ascertain afflux; loss of head curves.

Design. Comments on a Few Dams and Reservoirs, C. E. Grunsky. *Military Engr.*, vol. 23, no. 129, May-June 1931, pp. 220–228, 5 figs. Failure of Lafayette Dam; spillway capacity of storage dams; rain intensity and run-off formulas.

Earth. Design and Construction of Two New Reclamation Dams. *Eng. News-Rec.*, vol. 106, no. 25, June 18, 1931, pp. 998–1002, 7 figs. Descriptions of Gibson concrete arch dam, 25 miles northwest of Augusta, Mont., maximum height of 195 ft. and crest length 960 ft.; and of Echo earth-fill dam near Echo, Utah, about 30 miles northeast of Salt Lake City; maximum height 151 ft. above bedrock, with 25-ft. crest, 1,804 ft. long; elaborate equipment for testing pressure, temperature, and seepage in concrete dam.

Earth, Specifications. Cle Elum Dam, Yakima Project, Washington. *U.S. Bur. Reclamation—Specifications*, no. 522, 1931, 42 pp., 29 supp. plates. Specifications, schedules, and drawings for Cle Elum Dam, near Seattle; reservoir will furnish water for Yakima Project; main dam will consist of sprinkled and rolled earth-fill embankment, approximately 750 ft. long on crest with maximum height of 135 ft. above stream bed.

Hoover Dam Project. Hoover Dam Will Provide 1,000,000 Hp. *Power Plant Eng.*, vol. 35, no. 12, June 15, 1931, pp. 643–645, 2 figs. Great project of U.S. Bureau of Reclamation includes dam 730 ft. high at Black Canyon on Colorado River, power plant, diversion, and water control works, and other features; general plan and section of Hoover Dam, showing cofferdams and diversion tunnels.

Hoover Dam Project, Construction. Activities and Conditions at Boulder Dam. *Eng. News-Rec.*, vol. 106, no. 22, May 28, 1931, pp. 895–897, 4 figs. Chronicle of initial stage of construction; Las Vegas is frontier boom town; hundreds of job seekers; highway and railroad construction; contractor blasting road to tunnel portals; Boulder City plans; climatic conditions.

Hoover Dam Project, Power Line. Rushing the Power Line to Hoover Dam Site, R. H. Halpenny. *Eng. News-Rec.*, vol. 106, no. 26, June 25, 1931, pp. 1054–1057, 5 figs. Rugged desert country crossed in 225 days by 225-mile, 132-kv. transmission line from San Bernardino, Calif., to serve Hoover Dam construction activities; digging 7-ft. holes for tower bases; four erectors and two helpers fabricated each 52-ft. tower, piece by piece, from ground up; three conductors strung at same time.

Hydraulic Fill. Composition of Earth Dams—II. *Eng. News-Rec.*, vol. 106, no. 26, June 25, 1931, pp. 1044–1048, 3 figs. Symposium of three papers: Character of Available Material Fixes Core Dimensions, A. S. Crane; Slides During Construction Are Not Due to Core Pressure, F. H. Cothran; Major Settlement of Dam Cores Occurs During Construction, A. C. Eaton.

Hydraulic Gates. A Large Single-Faced Sluice. *Foundry Trade Journal*, vol. 44, no. 772, June 4, 1931, p. 792, 1 fig. Sluice supplied by Glenfield and Kennedy, for Pykara hydro-electric scheme, India, is 78 in. in diameter; constructed throughout of cast iron, with gunmetal faces.

Weirs, Discharge. Weir Discharge Chart, S. C. Mifflen. *Eng. and Min. Journal*, vol. 131, no. 10, May 25, 1931, p. 459. Chart designed to show quantity of water flowing over any one of three standard types of weir, when weir dimensions and head of water are known; formulas; example; procedure.

FLOOD CONTROL

Mississippi River. A New Mississippi Problem. *Eng. News-Rec.*, vol. 106, no. 25, June 18, 1931, p. 997. Editorial discussion of coming revision of plan to be considered by Congress; revision is being undertaken in utter ignorance of essential economic facts. Congress, in its action on flood control during coming session, should make such correction its first demand upon its engineers.

Drainage Works for the New Madrid Floodway, T. T. Knappen. *Eng. News-Rec.*, vol. 106, no. 22, May 28, 1931, pp. 878–882, 9 figs. Description of new Madrid, Mo., floodway drainage works in Mississippi River basin; construction of back levee across natural drainage courses necessitates ditch system, culverts, and bridges costing $500,000; structures, construction methods, and cost averages; Brewers Lake basin drainage; main drainage-ditch construction; levee crossings; bridge location; structures are standardized.

Rain and Rainfall, Run-Off. Storm-Water Discharge from Small and Large Areas—VII, H. C. Granville. *Indian Eng.*, vol. 89, no. 20, May 16, 1931, pp. 432–434, 9 figs. Three diagrams to numerical examples published in previous installments. (Continuation of serial.)

Storm-Water Discharge from Small and Large Areas—X, H. C. Granville. *Indian Eng.*, vol. 89, no. 21, May 23, 1931, pp. 454–456. Case of multiple of sand catchment; discussion of run-off formulas; summary.

FLOW OF FLUIDS

Fluid Motion. Some Problems Connected with Fluid Motion, J. J. Green. *Eng. Journal*, vol. 14, no. 6, June 1931, pp. 351–357, 10 figs. Development of theory of fluid flow; important results of Prandtl's hypothesis postulating existence of thin layer of fluid close to body, in which frictional force between adjacent fluid layers is proportional to velocity gradient normal to direction of motion; result of addition of viscous term by Stokes and of Prandtl's modification for boundary layer. Before Eng. Inst. Can.

Gases, Measurement. The Measurement of a Rapidly Fluctuating Flow of Gas, J. G. King and B. H. Williams. *Engineering*, vol. 131, no. 3413, June 12, 1931, pp. 759–760, 6 figs. In making water gas, process of generation is interrupted at short intervals, between which flow of gas is subject to rapid fluctuations; in investigations at Fuel Research Station, it was found necessary to obtain records of total volume flowing through main in given time, and of instantaneous rate of flow at any moment; apparatus used and results obtained during its calibration. From Dept. Sci. and Indus. Research, Fuel Research—Tech. Paper, no. 27. Price 6d. net.

Heat Transmission, Liquids in Tubes. An Equation for Heat in Fluids Flowing in Pipes, T. Bosch. *Mech. Eng.*, vol. 53, no. 7, July 1931, p. 541, 1 fig. Formulas of Prandtl; empirical formula derived by Eagle and Ferguson from their tests; translation of abstract previously indexed from V.D.I. Zeit. Jan. 10, 1931.

Pumps, Centrifugal. On Potential Flow of Water Through a Centrifugal Impeller, S. Uchimaru and S. Kito. *Tokyo Imperial Univ.—Faculty of Eng.—Journal*, vol. 19, no. 8, May 1931, pp. 191–223, 22 figs. Formulas for flow from single discharge source, around origin, from vortex source; form of vane curve adopted; motion of water at center of impeller.

FOUNDATIONS

Concrete. Monolith Foundations, H. C. Reid. *Junior Inst. Engrs.—Journal*, vol. 41, pt. 8, May 1931, pp. 337–349, 10 figs. Methods of constructing concrete foundation 40 ft. sq. and 100 ft. deep.

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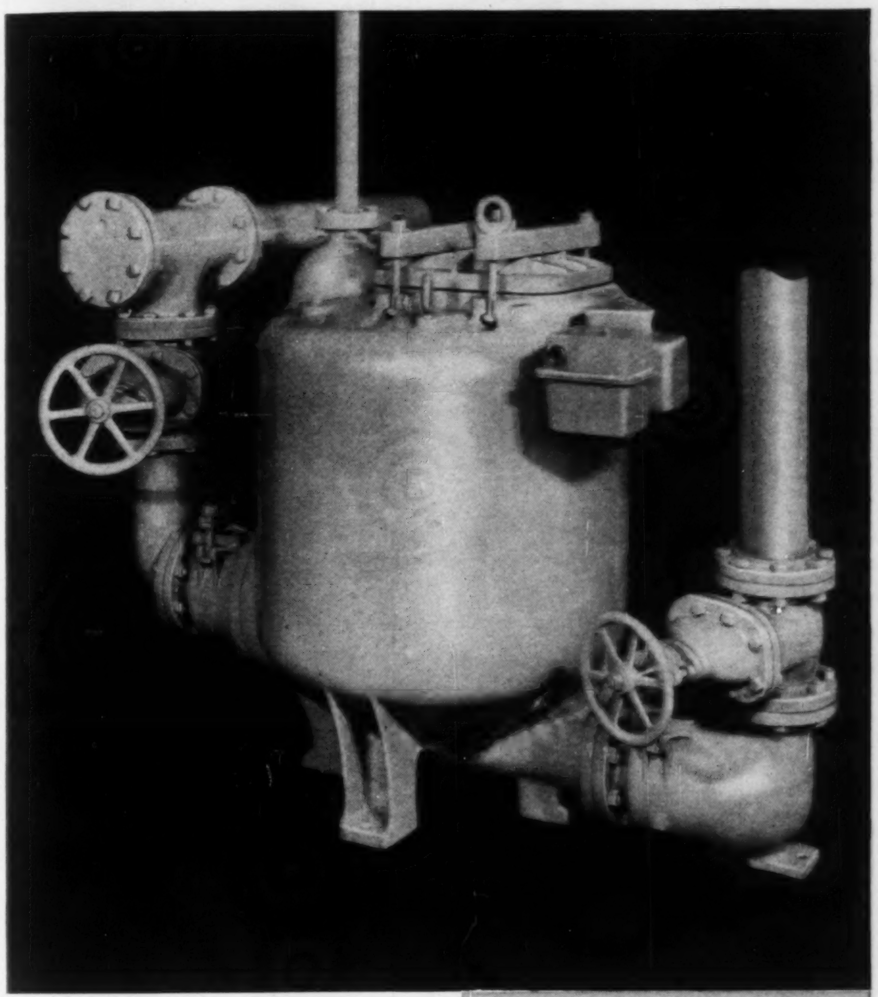

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Piles. Piles and Pile Foundations, J. S. Crandall. *Boston Soc. Civil Engrs.—Journal*, vol. 18, no. 5, May 1931, pp. 176–189, 4 figs. Pile-driving formulas; piles in silt and clay; soil loading due to individual and group piles.

HYDRO-ELECTRIC POWER PLANTS

Canada. Construction Methods on the Alexander Development, T. H. Hogg. *Contract Rec.*, vol. 45, no. 20, May 20, 1931, pp. 577–582, 6 figs. Description of 54,000-hp. development by Hydro-Electric Power Commission of Ontario on Nipigon River; unusual foundation conditions, dewatering by diversion channel, and scheme for closing this channel on completion of work; main dam of earth-fill construction has maximum height of 90 ft. (To be continued.)

Ontario. Alexander Power Development on the Nipigon River, T. H. Hogg. *Hydro-Elec. Power Commission Ont.—Bul.*, vol. 18, no. 5, May 1931, pp. 161–172, 10 figs. Paper before Eng. Inst. Can., previously indexed from various sources.

INLAND WATERWAYS

Canal Locks, Michigan. Locks at Sault Ste. Marie, Michigan, L. S. Dillon. *Military Engr.*, vol. 23, no. 129, May-June 1931, pp. 205–207, 6 figs. Early history and present conditions.

Canals, Welland. The Welland Ship Canal—XX. *Engineering*, vol. 131, no. 3414, June 19, 1931, pp. 787–790, 28 figs., partly on supp. plates. Particulars of 11 vertical-lift bridges on Welland Canal, varying from 208 ft. 8 in. between bearings, to 232 ft. $10^1/_2$ in.; all are square-built structures, with exception of Bridge No. 13 at Welland, which is skew to extent of 22 deg. 24 min. 30 sec.

The Welland Ship Canal—XX. *Engineering*, vol. 131, no. 3415, June 26, 1931, pp. 820–825, 19 figs. Machinery of vertical-lift bridges; since bridges are all located away from canal locks, 550-volt, 3-phase current for operating them is supplied from special substations which are described.

United States. What Savings in Transportation Are Made on Inland Waterways. *Ry. Age*, vol. 90, no. 21, May 23, 1931, pp. 1019–1020. Some widely quoted figures; letter from *Railway Age* to Maj.-Gen. Lytle-Brown, Chief of Army Engineers, regarding these figures, and General Brown's significant reply.

IRRIGATION

India. Irrigation in India—1928–29. *Engineer*, vol. 151, no. 3937, June 26, 1931, p. 711. During year, total area irrigated by government works of all classes in British India was 30.7 million acres; area irrigated was largest in Punjab; works completed and progress; projected works.

Java. Irrigation in Java, H. E. Babbitt. *Eng. News-Rec.*, vol. 107, no. 1, July 2, 1931, pp. 13–16, 9 figs. General description of island and its irrigation projects; excavation procedure on Tjisadame River Dam in which excavated material is carried off in stream; large irrigation projects under construction in Java; Patial River Dam.

MATERIALS TESTING

Clay Testing. The Effect of Thermal Shock on Clay Bodies, W. R. Morgan. *Univ. Ill.—Eng. Experiment Station—Bul.*, no. 229, no. 28, no. 43, June 23, 1931, pp. 5–27, 17 figs. Method for making comparisons of resistance of bodies to thermal shock on quantitative basis; relations between physical properties and resistance to thermal shock by means of which resistant bodies could be designed and plant control of their manufacture effected.

Concrete, Permeability. New Permeability Specimen for Comparison Tests, C. M. Chapman. *Eng. News-Rec.*, vol. 107, no. 1, July 2, 1931, p. 9, 1 fig. Description of newly devised form of test specimen for rating permeability of concrete, which is claimed to combine cheapness, ease of molding and testing, and uniformity.

Friction Testing. Friction Tests on Bearing Plate Materials, T. E. Stanton. *Eng. News-Rec.*, vol. 106, no. 26, June 25, 1931, pp. 1058–1060, 5 figs. Report on extensive tests to determine frictional properties of various materials for bridge bearing plates, recently carried on in the laboratory of the California Division of Highways; lead bronzes prove superior to harder tin alloys; a machine developed to duplicate field loading; results change California specifications.

Pavements, Concrete. Pavement Cores Stronger than Test Cylinders. *Eng. News-Rec.*, vol. 107, no. 1, July 2, 1931, p. 12. Recent data on concrete pavement cores and test cylinders obtained by Department of Public Works of Allegheny County (Pittsburgh).

Steel Fatigue. Failure of Machine Members, L. T. Holt. *Mach. Design*, vol. 3, no. 6, June 1931, pp. 41–43, 9 figs. Notch fatigue, insufficient heat treatment, and tool marks are considered as some of principal causes of failure.

United States. Developments in Materials and Testing. *Eng. News-Rec.*, vol. 107, no. 1, July 2, 1931, pp. 22–26. Report on proceedings of Chicago meeting of American Society for Testing Materials; abstracts of papers and discussions on use of specifications, specification changes, current research work, new test methods, corrosion, endurance strength, weathering of masonry, cement, and concrete progress; special papers on: rubber test model of dam; Mount Hope wire test; emulsified asphalt; abrasion testing of rubber; high-temperature effect on metals.

MUNICIPAL ENGINEERING

Dublin, Ireland. Greater Dublin, M. A. Moynihan. *Inst. Mun. and County Engrs.—Journal*, vol. 57, no. 25, June 9, 1931, pp. 1327–1342 and (discussion) 1343–1347, 1 fig. History of municipal improvements and description of streets, water works, street cleaning, housing, and sewage system.

Singapore. Municipal Engineering in Singapore, H. E. Babbitt. *Eng. News-Rec.*, vol. 106, no. 25, June 18, 1931, pp. 1009–1012, 5 figs. Survey of engineering activities of municipal government of Singapore; roads and streets; bridges; incinerator; town cleansing department; impounding reservoirs in Singapore water supply; water consumption in Singapore; water department; sewer system.

PORTS AND MARITIME STRUCTURES

Entrances, Great Lakes. Breakwater Construction for New Type of Great Lakes Harbor. *Eng. News-Rec.*, vol. 106, no. 25, June 18, 1931, pp. 1017–1019, 4 figs. Description of radical change in type of harbor entrance being made at Frankfort, Mich., and other ports on east shore of Lake Michigan; outer breakwaters replace piers and narrow channel; concrete caissons for main part; pile and concrete ends; collapsible form used on breakwater.

Ferry Landings. Train-Ferry Landings at Port Mulgrave and Point Tupper, N.S., D. B. Armstrong and W. C. Thomson. *Eng. Journal*, vol. 14, no. 6, June 1931, pp. 340–341. Discussion of paper before Eng. Inst. Can. previously indexed from issue of Jan. 1931.

RAILROADS, STATIONS, AND TERMINALS

Railroad Ballast. This Question of Ballast, J. V. Neubert. *Eng. and Maintenance*, vol. 27, no. 7, July 1931, pp. 639–640, 1 fig. Discussion of properties and life of ballast; ballast should be clean when applied; careful preparation of roadbed. Before Maintenance Way Club, Chicago.

ROADS AND STREETS

Machinery. Modern Road Surfacing Plant. *Road Surfacing and Maintenance*, vol. 36, no. 411, May 1931, pp. 26–27, 29, 31, 33, and 37, 11 figs. Review of recent innovations; spraying and gritting machines; road-rollers, scarifiers, and finishing plant; concrete mixers; macadam mixing plants.

Materials, Wire Mesh. The Use of Wire Mesh for Road Surfacing, J. G. Christiansen. *Military Engr.*, vol. 23, no. 129, May-June 1931, p. 237, 1 fig. Successful use of netting of no. 9 gage with 2-in. mesh galvanized after weaving width 8 ft., weight approximately 6 lb. per lineal foot, for temporary road in Alaska when ordinary surfacing materials were scarce.

SEWERAGE AND SEWAGE DISPOSAL

Disposal Plants, Great Britain. Exeter's New Sewage Purification Works, E. J. Silcock. *Surveyor*, vol. 79, no. 2054, pp. 603–604. Description of plant to use activated sludge method, for dry weather flow of 2,500,000 gal.

Joints. Pre-cast Asphaltic Sewer Joints Solve Wet Trench Problems, E. F. Chase. *Eng. News-Rec.*, vol. 106, no. 24, June 11, 1931, pp. 976–977, 3 figs. Report on practice of Department of Streets and Sewers, Seattle, Wash., with sewer joints of asphaltic rings pre-cast in bell and on spigot of pipe, for intercepting sewer where extreme groundwater conditions in clay and quicksand formation on pumping line made effective joints of extreme importance; manufacture of joints under plant condition by skilled workmen; advantages of new joint.

Outfall, South Africa. Extension of Green Point Sea Outfall, Cape Town, W. J. Houghton and H. C. Mason. *Surveyor*, vol. 79, no. 2051, May 15, 1931, pp. 531–532, 3 figs. Construction of multiple sewer outfall consisting of 27-in. steel pipes laid at depth of 50 ft.; calculations for wave pressure; anchorage; submerged pipes of mild steel, maximum length 25 ft., fitted with flanged joints bolted, and coated with $^5/_{16}$-in. of bitumen on inside; anchorages have double external wrapping impregnated with bitumen on outside. Before Inst. Municipal and County Engrs.

SURVEYING

City. Systematized Re-surveys of Cities, D. W. Bingham. *Mich. Engr.*, vol. 49, no. 2, June 1931, pp. 29–31. Discussion by C. H. Redman of paper previously indexed from issue of Mar. 1931.

Instruments. Inquiry into the Relative Accuracy of The Optical-Micrometer Theodolite, Briggs and Connor. *Inst. Min. Engrs.—Trans.*, vol. 81, pt. 2, May 1931, pp. 286–287; see also *Min. Inst. Scotland—Trans.*, vol. 52, pt. 1, Apr. 18, 1931, pp. 5–6. Discussion of paper previously indexed from various sources.

TRAFFIC CONTROL

Boston. Planning Boston's Thoroughfares, F. H. Fay. *Boston Soc. Civil Engrs.—Journal*, vol. 18, no. 5, May 1931, pp. 153–161. Study of traffic conditions; need for plan.

TUNNELS

Construction. Tunnel Engineering, J. C. Meen. *Professional Engr.*, vol. 16, no. 5, May 1931, pp. 9–11 and 29. Subject is considered under four heads: preliminary surveys and borings; design, line, and grade work; methods; and construction.

Rock Drills. Tunneling Equipment—I, Development of the Rock Drill, C. H. Vivian. *Eng. News-Rec.*, vol. 106, no. 25, June 18, 1931, p. 1024. Discussion by O. P. Erickson, of paper previously indexed from issue of Apr. 9, 1931.

Vehicular, Great Britain. The Mersey Tunnel, B. H. M. Hewitt. *Civil Eng. (Lond.)*, vol. 25, no. 12, May 1931, p. 615. Report on sinking of two preliminary headings and on system of ventilation for vehicular tunnel from Old Haymarket in Liverpool to Chester Street in Birkenhead; internal diam. of 44 ft. giving roadway width of 36 ft. between curves; length of tunnel, 2.93 miles; ruling gradient 1 in 30.

WATER PIPE LINES

Pipe Corrosion. Soil Corrosion Studies Reveal Important Facts, J. E. Schipper. *Water Works Eng.*, vol. 84, no. 12, June 17, 1931, pp. 890 and 905–906. Report on tests, under supervision of U.S. Bureau of Standards, which have been in progress for 10 years; selection of soil types; method of burying specimens; rate of corrosion changes.

Relocation. Difficulties Encountered and Methods Employed in Lowering a Main Under Pressure, C. M. Saville. *Water Works Eng.*, vol. 84, no. 10, pp. 653–654, and 729, 6 figs. Experience of Water Bureau of Metropolitan District of Hartford, Conn., in lowering 30-in. cast-iron main from $2^1/_2$ to 8 ft.; easier work let by contract, more difficult done by department hand labor; method of taking care of house sewer connections; lowering portion with vertical curve and gate; cost of work; obsolescence.

Service Connections. Service Pipes of Various Materials, R. W. Reynolds. *Am. Water Works Assn.—Journal*, vol. 23, no. 5, May 1931, pp. 658–663 and (discussion) 663–672, 1 fig. Report on experience of West Palm Beach Water Company, with galvanized iron pipe service, cement lined galvanized service pipe, and with copper tubing; comparative tests of friction loss in service pipe of these three materials, also of used unlined iron pipe; adopted use of copper tubing as standard practice during 1929.

Welding. High-Pressure Pipe Lines, J. Kloepper and J. Wasser. *Eng. Progress*, vol. 12, no. 6, June 1931, pp. 121–127, 17 figs. Water-gas lap-welded pipe in its application of high-pressure gas and water lines; review of salient points of fabrication and properties of welded pipe.

WATER TREATMENT

Bacteriology. Elements of Water Bacteriology, with Special Reference to Sanitary Water Analysis, S. C. Prescott and C. E. A. Winslow. 5th edition. N.Y., John Wiley & Sons, 1931, 219 pp., tables. $2.50. Edition follows that of 1924 in general, but new procedures of value have been introduced and extensive bibliography has been brought up to date. This book has long been valued as a guide to best American practices, and is arranged to meet the needs of sanitary engineers, public hygienists, and others interested in water and sewage problems. Eng. Soc. Lib., N.Y.

Pollution. Insuring the Delivery of a Safe Water to the Consumer, A. H. Wieters. *Water Works Eng.*, vol. 84, no. 3, Feb. 11, 1931, pp. 187–188. Responsibility of water works superintendent for supply; the various causes of pollution, needed safeguards, cross connections within factories, dangerous plumbing fixtures, and the danger in laying new mains, are entirely and carefully covered.

WATER WORKS ENGINEERING

Colorado River. Features of the Colorado River Aqueduct, J. Hinds. *West. City*, vol. 7, no. 6, June 1931, pp. 13–16, 3 figs. Plans contemplate diversion of continuous flow of 1,500 sec.-ft.; total length of route 267 miles; examination of gravity routes; necessity for pumping project; Parker power plant to generate 80,000 kw. Before Nat. Convention, Am. Water Works Assn.

Water Works Symposium. The Institution of Water Engineers. *Engineering*, vol. 131, no. 3415, June 26, 1931, pp. 837–838. Review of papers read at meeting, June 17, 1931: Recent Water Legislation, H. P. Hill; Dam Construction, C. M. Saville; Construction of Water Mains, F. W. Macaulay; Water Purification, E. S. Chase; Sinking of Borings, H. A. P. Hetherington.